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December 19, 1996

GRI Report #18

Cover Soil Slope Stability Involving Geosynthetic Interfaces

Errata

Section 3.2 - Incorporation of Equipment Loads

Page 22 Equation 17 should be $F_e = W_e \left(\frac{a}{g}\right)$

Page 23 Equation 19 should be $N_e = W_e \cos\beta$

Page 81 Equation for F_e should be $F_e = W_e\,(a/g) = 0.0$

Page 82 Equation for F_e should be $F_e = W_e\,(a/g) = 16.6$

Page 82 the resultant FS should be 1.02

Comments:

(a) For hand calculations, the corrections to equations 17 and 19 should suffice.

(b) For spread sheet calculations, the "EQUIPMNT.GRI" spread sheet has to be modified. Simply go to the cell which stores the function for "F_e" calculation (cell I39) and delete the terms corresponding to the multiplication of the influence factor. (i.e., delete "*B40")

We apology for the inconvenience. Please let us know should you have any further questions.

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COVER SOIL SLOPE STABILITY INVOLVING GEOSYNTHETIC INTERFACES

by

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GRI Report #18

December 9, 1996

COVER SOIL SLOPE STABILITY INVOLVING GEOSYNTHETIC INTERFACES

Table of Contents

Abstract

While soil slope failures are undoubtedly ages old, it was not until 1905 that a rigorous and analytic attempt was formulated and found to be generally acceptable. This attempt was prompted by a massive hillside failure in Sweden carrying hundreds of train-riding vacationers to their deaths. The subsequent commission, led by A. Atterberg, developed the Swedish slip circle method. Over subsequent years, the technique was modified to include effective stresses and has been incrementally refined by various researchers, e.g.,

- Bishop method,
- modified Bishop method,
- Janbu method, and
- Morgenstern-Price method.

The situation has progressed to the point where soil slope failures in geotechnical engineering applications are relatively uncommon. What failures that do occur are usually shallow surface sloughs or result from errors or misjudgment in design or testing. Attesting to this relatively established situation is the gradual decrease in acceptable factors of safety (FS) values. It is not uncommon to review designs with values as low as 1.2 for temporary/noncritical situations as being acceptable and constructed accordingly.

The emergence of solid waste landfills, abandoned dumps and remediated waste piles, however, has changed the situation considerably. Cover soil instability in the liner system beneath the waste mass, as well as in the cover system above the waste mass occurs far too frequently. The reasons are obvious:

(a) The slopes tend to be steep so as to maximize waste volume within a limited land area.

(b) The slopes tend to be long so as to maximize waste volume within a limited land area.

(c) Liner and/or cover systems contain anywhere from 1 to 12 geosynthetic interfaces.

(d) The orientation of the geosynthetic interfaces is exactly in the direction of an incipient slide.

Thus it is felt that a comprehensive report on liner and cover soil slope stability, based on limit equilibrium procedures, is appropriate.

The approach taken is in the form of a design/analysis tutorial. It begins with the basic problem of a finite length, uniform thickness cover soil on a geosynthetic surface, typically a geomembrane. The realism of including equipment loads is then analyzed. Bulldozer movement placing cover soil up the slope, then downward, is analyzed. Two methods to increase slope stability are then developed; tapered thickness cover soils and veneer reinforcement. Such reinforcement can be provided by geogrids or high strength geotextiles. Conversely, two phenomena that decrease slope stability are then developed; seepage forces and seismic excitation.

These situations are all examined in a theoretical manner with the requisite equations being developed and presented. A design chart using typical values is then developed. Each situation is then embellished with an example problem using the design charts. Holding as many variables constant as possible, the resulting FS-values for the various example problems are as follows.

Example	Condition	FS-value
1	standard example*	1.25
2(a)	equipment up-slope	1.24
2(b)	equipment down-slope	1.03
3	tapered thickness	1.57
4	veneer reinforcement	1.57
5	seepage	0.93
6	seismic	0.94

*30 m long slope, sand cover soil of 300 mm thickness, $\phi = 30$ deg., on a geomembrane with $\delta = 22$ deg., at a slope angle $\beta = 18.4$ deg.

The FS-values speak for themselves. Clearly, equipment moving down slopes, seepage forces and seismic excitation can be devastating. Conversely, tapered thickness cover soils or veneer reinforcement with geogrids or high strength geotextiles are seen to significantly increase the FS-values. The amount of the increase is at the discretion of the design engineer.

The situation of multi-lined slopes, with weakest interface located lower than the upper geosynthetic surface has many possible variations. Insight into typical problems of this type are give in Appendix "A".

Computer worksheets for each of the problems are included as Appendix "B". They result in design curves for the various example problems that were selected. These worksheets can be used for a wide variety of additional variables different from the ones that were arbitrarily selected by the authors.

Once the nuances of the analytic work are understood and appreciated, one must always challenge the realism of the laboratory interface shear test results which are necessarily used in the analysis. It does little good to have an analytic technique accurate to many decimal places, when one is selecting relatively loosely defined shear strength values from the literature. Instead, complete geotechnical simulation of the site specific conditions for the laboratory shear tests is necessary. This means that saturation conditions, hydrating liquid, testing temperature, normal stresses, consolidation duration, shearing rates, ultimate shearing deformation, etc., must accurately portray the site specific conditions. At this point one can have confidence in the laboratory test values to be used in the analytic formulation and hence the resulting FS-values. Unfortunately, such accurate shear strength testing is currently felt to be the weakest element of the entire process.

Acknowledgments

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COVER SOIL SLOPE STABILITY INVOLVING GEOSYNTHETIC INTERFACES

With increasing needs to maximize landfill air space (by virtue of economics, logistics, politics, etc.), the slopes of final covers of engineered landfills, abandoned dumps and remediated waste piles tend toward being relatively steep, rather than being relatively flat. Slopes of 3(H) to 1 (V), i.e., 18.4 deg. with the horizontal, are somewhat common and 2(H) to 1(V), i.e., 26.6 deg. with the horizontal, have also been utilized. The same situation occurs with drainage soils placed on geomembrane lined slopes beneath the waste of engineered landfills and heap leach mining operations. The exception being that these applications are relatively short-term situations which cease to be a concern after solid waste is placed against them along with the accompanying buttressing action provided. Couple these steep slopes with the realization that low interface shear strength inclusions (such as geomembranes, hydrated geosynthetic clay liners (GCLs) and/or wet-of-optimum compacted clay liners (CCLs)) are oriented precisely in the direction of a potential slide and the necessity for careful slope stability analyses should be obvious. Hence, the reason for this report.

1.0 Overview

With geomembranes, hydrated GCLs and/or wet-of-optimum CCLs used as barrier layer components in cover and liner situations, potential shear planes exist at a number of interfaces. Even further, geosynthetic drainage systems (for water drainage above and gas transmission below) can be involved with additional potential shear planes. A number of slides parallel to the slope angle have occurred and have been reported in the open literature. The most common situations appear to be the following:

- Cover soil sliding off the upper surface of a smooth geomembrane.
- Cover soil with an underlying geotextile or drainage geocomposite sliding off the upper surface of a smooth geomembrane.

- Cover soil, drainage materials and underlying geomembrane sliding off the upper surface of the underlying soil, e.g., a wet-of-optimum CCL.
- Cover soil, drainage materials and underlying geomembrane sliding off the upper surface of a underlying hydrated GCL, particularly if the upper surface of the GCL is a woven slit film geotextile.

From the perspective of a slope stability analysis, the actual or potential shear plane is generally linear, parallel to the slope angle and along the inclusion having the lowest interface shear strength. Thus, the analysis is straightforward and is felt to be within the state-of-the-practice. The analyses used herein are based upon limit equilibrium principles, however, it should be recognized that finite element methods have also been used for the same class of problems, see Wilson-Fahmy and Koerner (1993). Limit equilibrium analysis is a methodology which requires material-specific shear strength properties which are obtained from laboratory tests simulating the site-specific situation as closely as possible. In this regard, the results of direct shear tests will be seen to be the most significant input property in the analysis.

The result of slope stability analyses of the type to be described in this report is a global factor-of-safety (FS). This FS-value must be viewed in light of the significance of a potential failure. For example, the drainage system on a geomembrane beneath the waste might be designed for a value of 1.2 to 1.3, while a final cover under similar circumstances would generally require a value of 1.4 to 1.5. This, of course, depends upon site specific conditions and (usually) a review by the appropriate regulatory agency.

In the sections to follow in this report, the general principles of limit equilibrium analysis will be presented. Details and nuances of direct shear testing will be included. Various slope stability scenarios will follow, with details on both constant thickness cover soils and live load considerations. To offset potentially low FS-values, procedures using tapered thickness cover soils and/or using geosynthetic reinforcement will be presented. Seepage considerations will form a separate section, as will seismic considerations. The summary will give design suggestions on approaches to minimize slope stability concerns of final covers and some alternative strategies. It will also present the authors' recommendations for minimum FS-values under different waste-specific and risk-specific situations. Two appendices will be provided. One deals with multiple interfaces with a low shear strength material beneath the uppermost interface. The other gives computer worksheets for calculating FS-values for each of the classes of problems that were evaluated.

2.0 Geotechnical Principles and Issues

As mentioned previously, the potential failure surface for final covers is usually linear with an overlying cover soil sliding with respect to the lowest interface friction layer in the underlying cross section. The potential failure plane being linear allows for a straightforward calculation without the need for trial center locations and different radii as with soil slopes analyzed by rotational failure surfaces. Furthermore, full static equilibrium can be achieved without solving simultaneous equations or making simplified design assumptions.

2.1 Limit Equilibrium Concepts

The free body diagram of an *infinitely long slope* with uniformly thick cohesionless cover soil on an incipient planar shear surface, like the upper surface of a geomembrane, is shown in Figure 1. The situation can be treated quite simply.

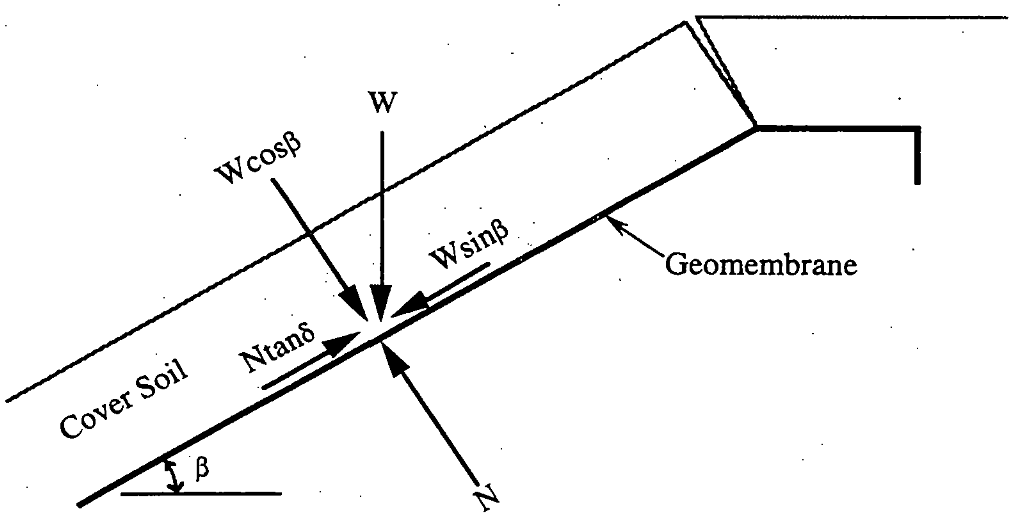


Figure 1 - Limit equilibrium forces involved in an infinite slope analysis for a uniformly thick cohesionless cover soil

By taking force summation parallel to the slope and comparing the resisting force to the driving or mobilizing force, a global factor-of-safety (FS) results.

$$FS = \frac{\sum \text{Resisting Forces}}{\sum \text{Driving Forces}}$$

$$= \frac{N \tan\delta}{W \sin\beta} = \frac{W \cos\beta \tan\delta}{W \sin\beta}$$

Hence

$$FS = \frac{\tan\delta}{\tan\beta} \tag{1}$$

Here it is seen that the FS-value is the ratio of tangents of the interface friction angle of the cover soil to the upper surface of the geomembrane (δ), and the slope angle of the soil beneath the geomembrane (β). As simple as this analysis is, its teachings are very significant, for example:

- To obtain an accurate FS-value, an accurately determined laboratory δ-value is absolutely critical. The accuracy of the analysis is only as good as the accuracy of the laboratory obtained δ-value.
- For low δ-values, the resulting soil slope angle will be proportionately low. For example, for a δ-value of 20 deg., and a required FS-value of 1.5, the maximum slope angle is 14 deg. This is equivalent to a 4(H) on 1(V) slope which is relatively low. Furthermore, many geomembranes have even lower δ-values, e.g., 10 to 15 deg.
- This simple formula has driven geosynthetic manufacturers to develop products with high δ-values, i.e., textured geomembranes, thermally bonded drainage geocomposites, internally reinforced GCLs, etc.

Unfortunately, the above analysis is too simplistic to use in most practical situations. For example, the following situations cannot be accommodated;

- a finite length slope with the incorporation of a passive soil wedge in the analysis,
- the incorporation of equipment loads on the slope,
- the use of tapered cover soils thickness,

- veneer reinforcement of the cover soil using geogrids or high strength geotextiles,
- consideration of seepage forces in the cover soil, or
- consideration of seismic forces acting on the cover soil.

These situations will be treated in subsequent sections. For each situation, the essence of the theory will be presented, followed by the necessary design equations. This will be followed, in each case, with a design chart and an example problem. First, however, the issue of interface shear testing will be discussed.

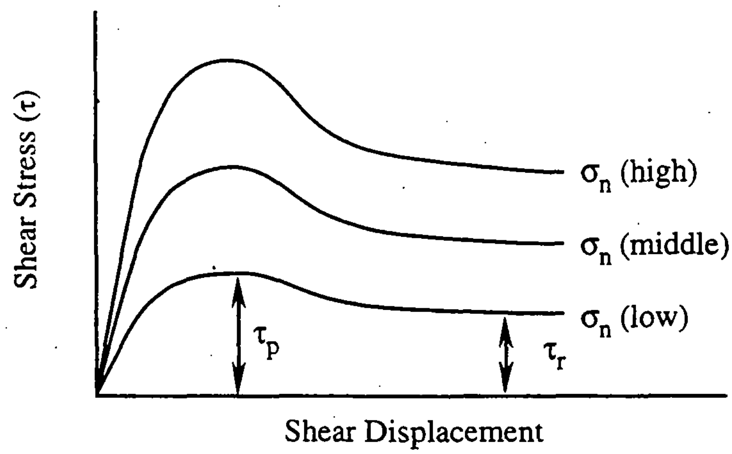
2.2 Interface Shear Testing

The interface shear strength of a cover soil with respect to the underlying material (often a geomembrane) is critical to properly analyze the stability of the cover soil. This value of interface shear strength is obtained by laboratory testing of the project specific materials at the site specific conditions. By project specific materials, we mean sampling of the candidate geosynthetics to be used at the site, as well as the cover soil at its targeted density and moisture content. By site specific conditions, we mean testing at the anticipated normal stresses, moisture conditions, temperature extremes (high and/or low), strain rates and total deformation values. *Note that it is completely inappropriate to use values of interface shear strengths from the literature for final cover design.*

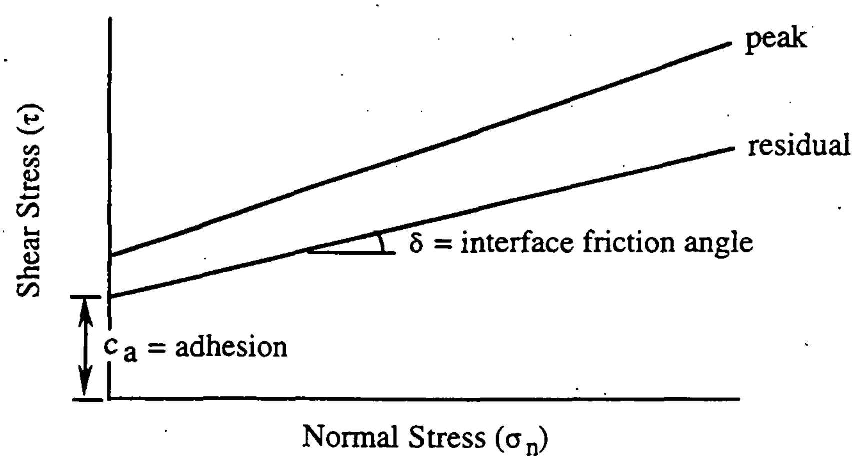
While the above list of items is formidable, at least the type of test is established. It is the direct shear test which has been utilized in geotechnical engineering testing for many years. The test has been adapted to evaluate geosynthetics in the USA and Germany as ASTM D5321 and DIN 60500, respectively.

In conducting a direct shear test on a specific interface, one typically performs three replicate tests with the only variable being three different values of normal stress. The middle value is usually targeted to the site specific condition, with a lower and higher values of normal stress covering the range of possible values. These three tests result in a set of shear displacement versus shear stress curves, see Figure 2(a). From each curve, a peak shear strength

(τ_p) and a residual shear strength (τ_r) is obtained. As a next step, these shear strength values, together with their respective normal stress values, are plotted on Mohr-Coulomb stress space, as in Figure 2(b).



(a) Direct shear test data



(b) Mohr-Coulomb stress space

Figure 2 - Direct shear test results and method of analysis to obtain shear strength parameters

The points are then connected (usually with a straight line), and the two fundamental shear strength parameters are obtained. These shear strength parameters are:

δ = the angle of shearing resistance, peak and/or residual, of the two opposing surfaces (generally called the interface friction angle)

c_a = the adhesion of the two opposing surfaces, peak and/or residual (synonymous with cohesion when testing fine grained soils against one another)

These two parameters constitute the equation of a straight line which is the Mohr-Coulomb failure criterion common to geotechnical engineering. The concept is readily adaptable to geosynthetic materials in the following form:

$$\tau_p = c_{ap} + \sigma_n \tan \delta_p \qquad (2a)$$

$$\tau_r = c_{ar} + \sigma_n \tan \delta_r \qquad (2b)$$

The upper limit of "δ_p" when soil is involved as one of the interfaces is "ϕ", the angle of shearing resistance of the soil component. The upper limit of the "c_{ap}" value is "c", the cohesion of the soil component. In the slope stability analyses to follow, the "c_{ap} or c_{ar}" term, if one is present, will not be utilized. There must be a clear physical justification for use of such values when geosynthetics are involved. Only unique situations such as textured geomembranes with physical interlocking, or the bentonite component of a GCL are valid reasons for including such a term.

To be noted is that residual strengths are equal, or lower, than peak strengths. The amount of difference is very dependent on the material and no general guidelines can be given. Clearly, project-specific and material-specific direct shear tests must be performed to determine the appropriate values. Further, each direct shear test must be conducted to an adequate displacement to determine the residual behavior, see Stark and Poeppel (1994). The decision as to the use of peak or residual strengths in the subsequent analysis is a very subjective one. It is clearly a site specific and materials specific issue which is left up to the designer and/or regulator. Even further, the use of peak values at the crest of the slope and residual values at the toe may be justified. As such, the analyses to follow will use an interface δ-value with no subscript thereby concentrating on the computational procedures rather than this particular detail. However, the importance of the appropriate and accurate δ-value should not be minimized.

Due to the physical structure of many geosynthetics, the size of the recommended shear box is quite large. It must be at least 300 mm by 300 mm unless it can be shown that data generated by a smaller device contains no scale or edge effects, i.e., that no bias exists with a smaller shear box. The implications of such a large shear box should not be taken lightly. Some issues which should receive particular attention are the following:

- Unless it can be justified otherwise, the interface will usually be tested in a saturated state. Thus complete and uniform saturation over the entire area must be achieved. This is particularly necessary for GCLs, Daniel, et al. (1993). Hydration takes relatively long in comparison to soils in conventional (smaller) testing shear boxes.
- Consolidation of soils (including CCLs and GCLs) in large shear boxes is similarly effected.
- Uniformity of normal stress over the entire area must be maintained during consolidation and shearing so as to avoid stress concentrations from occurring.
- The application of relatively low normal stresses, e.g., 10, 20 and 30 kPa simulating typical cover soil thicknesses, challenges the accuracy of some commercially available shear box setups and monitoring systems, particularly the accuracy of pressure gages.
- Shear rates necessary to attain drained conditions (if that is the desired situation) are extremely slow, requiring long test times.
- Deformations necessary to attain residual strengths require large relative movement of the two respective halves of the shear box. So as not to travel over the edges of the opposing shear box sections, devices should have the lower shear box larger than 300 mm. However, with a lower shear box larger than the upper traveling section, new surface is constantly being added to the shearing plane. This influence is not clear in the response or in the subsequent behavior.

- The attainment of a true residual strength is difficult to achieve. ASTM D5321 states that one should "run the test until the applied shear force remains constant with increasing displacement". Many commercially available shear boxes have insufficient travel to reach this condition.
- The ring torsion shearing apparatus is an alternative device to determine true residual strength values, but is not without its own problems. See Stark and Poeppel (1994) for information and data using this alternative test method.

2.3 Various Situations Encountered

There is a large variety of slope stability problems that may be encountered in analyzing and/or designing final covers of engineered landfills, abandoned dumps and remediation sites as well as drainage soils covering geomembranes beneath the waste. Perhaps the most common is a uniformly thick cover soil on a geomembrane covering the slope at a given and constant slope angle. This "standard" problem will be analyzed in the next section. A variation of this problem will include equipment loads used during placement of cover soil on the geomembrane. This problem will be solved with equipment moving up the slope and then moving down the slope.

When low FS-values arise in the above problems, the designer has a number of options. Other than a geometric redesign of the slope, there are two options commonly used. These are to use a tapered cover soil thickness and/or and the use of geosynthetic reinforcement. This latter option is called "veneer reinforcement" in the literature and comes about by the inclusion of a geogrid or high strength geotextile within the cover soil. Both of these situations will be illustrated.

Unfortunately, cover soil failures have occurred and perhaps the majority of the failures have been associated with seepage forces. Indeed, drainage above a geomembrane (or other barrier material) in the cover soil cross section must be accommodated to avoid the possibility of seepage forces. A section will be devoted to this class of slope stability problems.

Lastly, the possibility of seismic forces exists in earthquake prone locations. If an earthquake occurs in the vicinity of an engineered landfill, abandoned dump or remediation site, the seismic wave travels through the solid waste mass reaching the upper surface of the cover. It then decouples from the cover soil materials, producing a horizontal force which must be appropriately analyzed. A section will be devoted to the seismic aspects of cover soil slope analysis as well.

A subset of all of these problems are cases of multi-lined slopes. The design goal in such cases is to have all interface friction angles higher than the slope angle, hence stability is assured, recall equation (1). Unfortunately, this is sometimes not possible or practical. A variety of such situations will be addressed in Appendix "A".

3.0 Cover Soil Slope Stability Problems

This section presents the analytic formulations, design curves and example problems of a number of common slope stability problems. The standard problem of a uniformly thick cover soil is developed without, then with, equipment loading. When the resulting FS-value is too low the designer can select a number of options. For example, lowering of the slope angle, reduction of the slope length with intermediate berms or the use of higher shear strength materials are possible design options. The analysis procedure is the same regardless of these decisions. Quite different strategies are the use of tapered cover soil thickness and/or the use of high strength geosynthetic inclusions. These two strategies will be developed later in this section as being fundamentally different and somewhat unique design alternatives.

3.1 Slopes with Uniformly Thick Cover Soils

Figure 3 illustrates a uniformly thick cover soil on a geomembrane at a slope angle "β" which is of finite length. It includes a passive wedge at the base and a tension crack at the top. The analysis that follows is after Koerner and Hwu (1991), but comparable analyses are available from Giroud and Beech (1989) and McKelvey and Deutsch (1991).

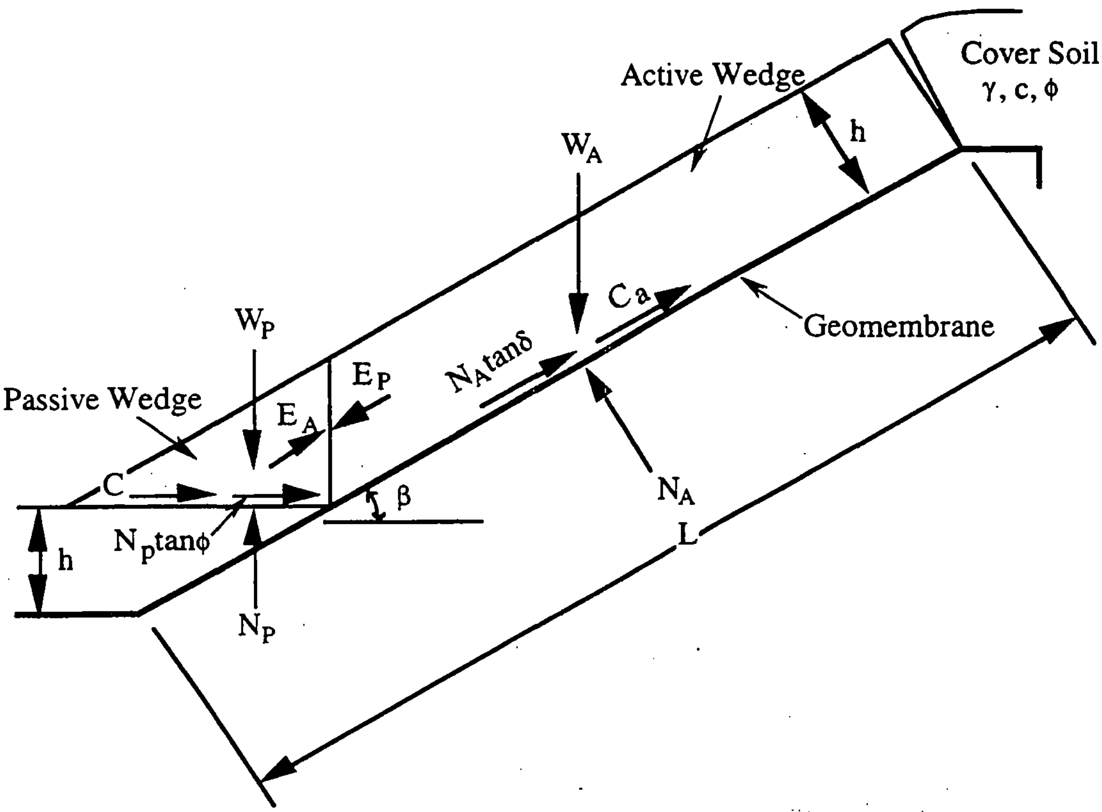


Figure 3 - Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil

The symbols used in Figure 3 are defined below.

W_A = total weight of the active wedge

W_P = total weight of the passive wedge

N_A = effective force normal to the failure plane of the active wedge

N_P = effective force normal to the failure plane of the passive wedge

γ = unit weight of the cover soil

h = thickness of the cover soil

L = length of slope measured along the geomembrane

β = soil slope angle beneath the geomembrane

ϕ = friction angle of the cover soil

δ = interface friction angle between cover soil and geomembrane

C_a = adhesive force between cover soil of the active wedge and the geomembrane

c_a = adhesion between cover soil of the active wedge and the geomembrane

C	= cohesive force along the failure plane of the passive wedge
c	= cohesion of the cover soil
E_A	= interwedge force acting on the active wedge from the passive wedge
E_P	= interwedge force acting on the passive wedge from the active wedge
FS	= factor-of-safety against cover soil sliding on the geomembrane

The expression for determining the factor-of-safety can be derived as follows:

Considering the active wedge,

$$W_A = \gamma h^2 \left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2} \right) \tag{3}$$

$$N_A = W_A \cos\beta \tag{4}$$

$$C_a = c_a \left(L - \frac{h}{\sin\beta} \right) \tag{5}$$

By balancing the forces in the vertical direction, the following formulation results:

$$E_A \sin\beta = W_A - N_A \cos\beta - \frac{N_A \tan\delta + C_a}{FS} \sin\beta \tag{6}$$

Hence the interwedge force acting on the active wedge is:

$$E_A = \frac{(FS)(W_A - N_A \cos\beta) - (N_A \tan\delta + C_a)\sin\beta}{\sin\beta (FS)} \tag{7}$$

The passive wedge can be considered in a similar manner:

$$W_P = \frac{\gamma h^2}{\sin 2\beta} \tag{8}$$

$$N_p = W_P + E_P \sin\beta \tag{9}$$

$$C = \frac{(c)(h)}{\sin\beta} \tag{10}$$

By balancing the forces in the horizontal direction, the following formulation results:

$$E_P \cos\beta = \frac{C + N_P \tan\phi}{FS} \tag{11}$$

Hence the interwedge force acting on the passive wedge is:

$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi} \tag{12}$$

By setting $E_A = E_P$, the following equation can be arranged in the form of $ax^2+bx+c=0$ which in our case using FS-values is:

$$a(FS)^2 + b(FS) + c = 0 \tag{13}$$

where

$$a = (W_A - N_A\cos\beta)\cos\beta$$

$$b = -[(W_A - N_A\cos\beta)\sin\beta\tan\phi + (N_A\tan\delta + C_a)\sin\beta\cos\beta + \sin\beta(C + W_P\tan\phi)]$$

$$c = (N_A\tan\delta + C_a)\sin^2\beta\tan\phi \tag{14}$$

The resulting FS-value is then obtained from the following equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \tag{15}$$

When the calculated FS-value falls below 1.0, a stability failure of the cover soil sliding on the geomembrane is to be anticipated. Thus a value of greater than 1.0 must be targeted as being the minimum factor-of-safety. How much greater than 1.0 the FS-value should be, is a design and/or regulatory issue. The issue of minimum allowable FS-values under different conditions will be revisited at the end of this report.

In order to better illustrate the implications of equations 13, 14 and 15, typical design curves for various FS-values as a function of slope angle and interface friction angle are given in Figure 4. Note that the curves are developed specifically for the variables stated in the legend of the figure. Example problem #1 illustrates the use of the curves.

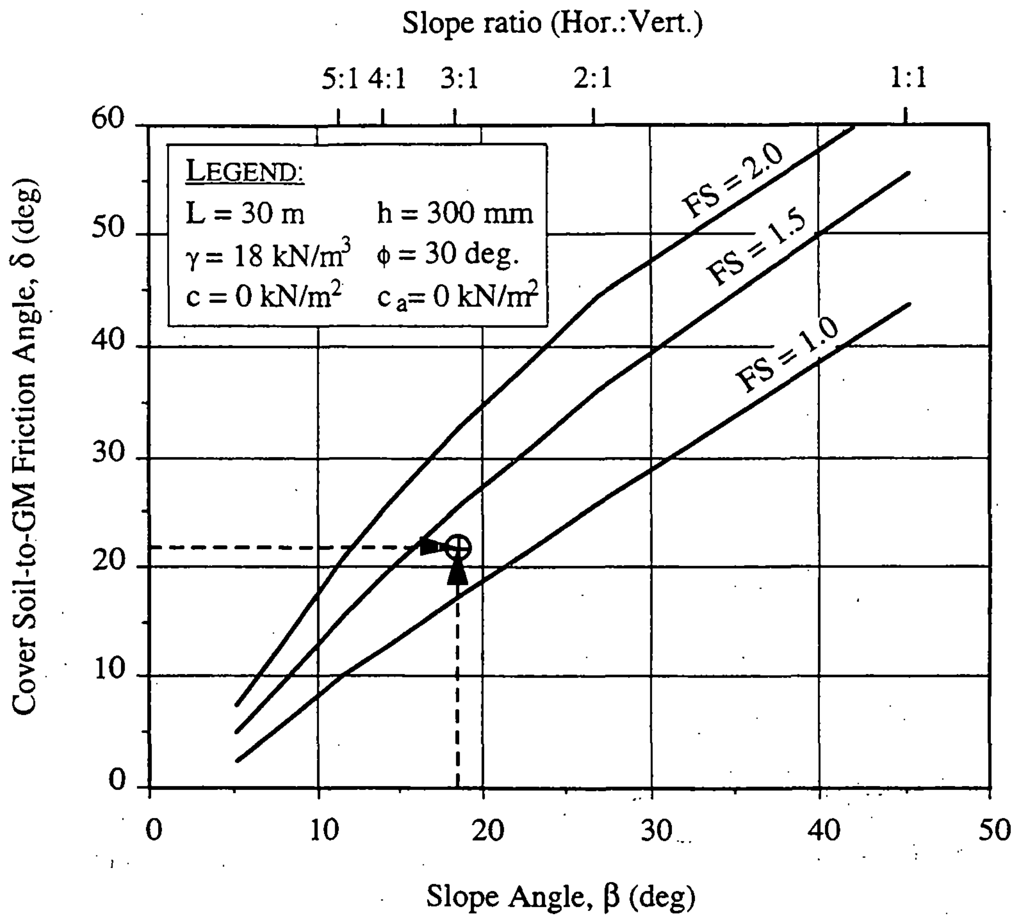


Figure 4 - Design curves for stability of uniform-thickness cohesionless cover soils on liner failure planes for various global factors-of-safety

<u>Example 1</u>: Given a 30 m long slope with a uniformly thick cover soil of 300 mm at a unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The cover soil is on a geomembrane as shown in Figure 3. Direct shear testing has resulted in a interface friction angle between the cover soil and geomembrane of 22 deg. and zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg?

<u>Solution</u>: Using the design curves of Figure 4 (which were developed for the exact conditions of the example problem), the resulting FS = 1.25.

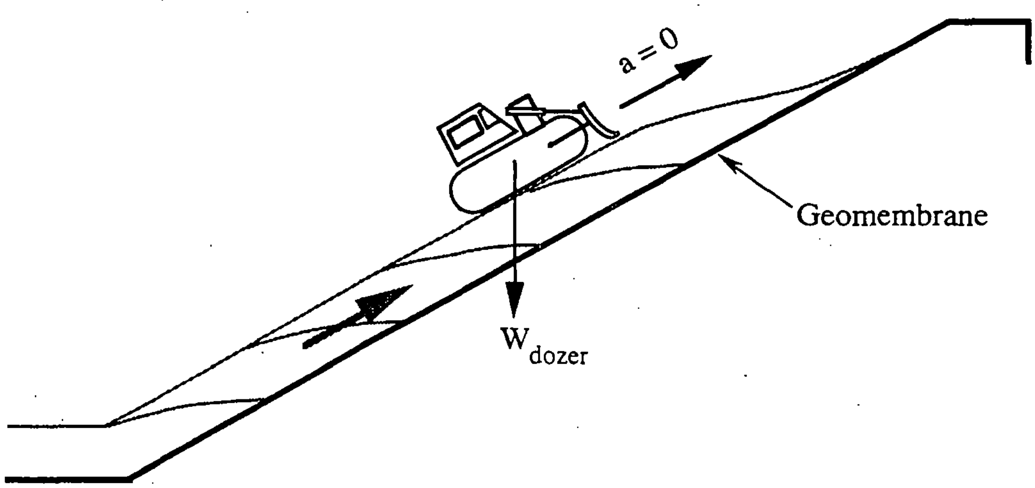
<u>Comment:</u> In general, this is too low of a value for a final cover slope factor-of-safety and a redesign is necessary. While there are many possible options of changing the geometry of the

situation, the example will be revisited later in this section using tapered cover soil thickness and veneer reinforcement. Furthermore, this general problem will be used throughout the main body of this report for comparison purposes to other cover soil slope stability situations.

3.2 Incorporation of Equipment Loads

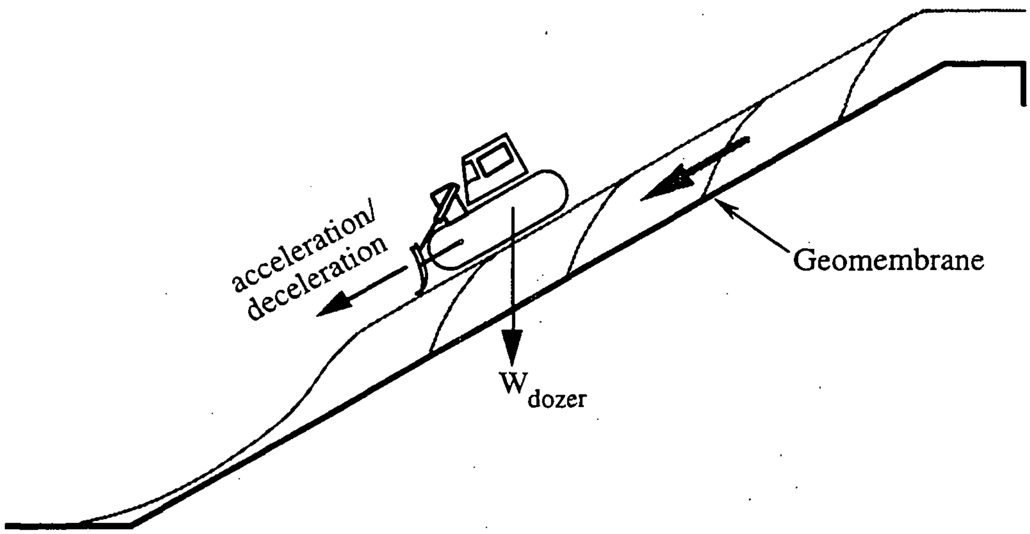
The placement of cover soil on a slope with a relatively low shear strength inclusion (like a geomembrane) should *always* be from the toe upward to the crest. Figure 5(a) shows the recommended method. In so doing, the gravitational forces of the cover soil and live load of the construction equipment are compacting previously placed soil and working with an ever present passive wedge and stable lower-portion beneath the active wedge. While it is prudent to specify low ground pressure equipment to place the soil, the reduction of the FS-value for this situation of equipment working up the slope will be seen to be nominal.

For soil placement down the slope, however, a stability analysis must add an additional dynamic stress into the solution. This stress decreases the FS-value and in some cases to a great extent. Figure 5(b) shows this procedure. Unless absolutely necessary, it is not recommended to place cover soil on a slope in this manner. If it is necessary, the design must consider the dynamic force of the specific type of construction placement equipment.



(a) Equipment backfilling up slope (the recommended method)

Figure 5 - Construction equipment placing cover soil on slopes containing geosynthetics



(b) Equipment backfilling down slope (method is not recommended)

Figure 5 - Construction equipment placing cover soil on slopes containing geosynthetics *(cont'd)*

For the <u>first case</u> of a bulldozer pushing cover soil up from the toe of the slope to the crest, the analysis uses the free body diagram of Figure 6(a). The analysis uses a specific piece of construction equipment (like a bulldozer characterized by its weight or ground contact pressure) and dissipates this force or stress through the cover soil thickness to the surface of the geomembrane. A Boussinesq analysis is used, see Poulos and Davis (1974). This results in an equipment force per unit width as follows:

$$W_e = q\,w\,I \tag{16}$$

where

W_e = equivalent equipment force per unit width at the geomembrane interface

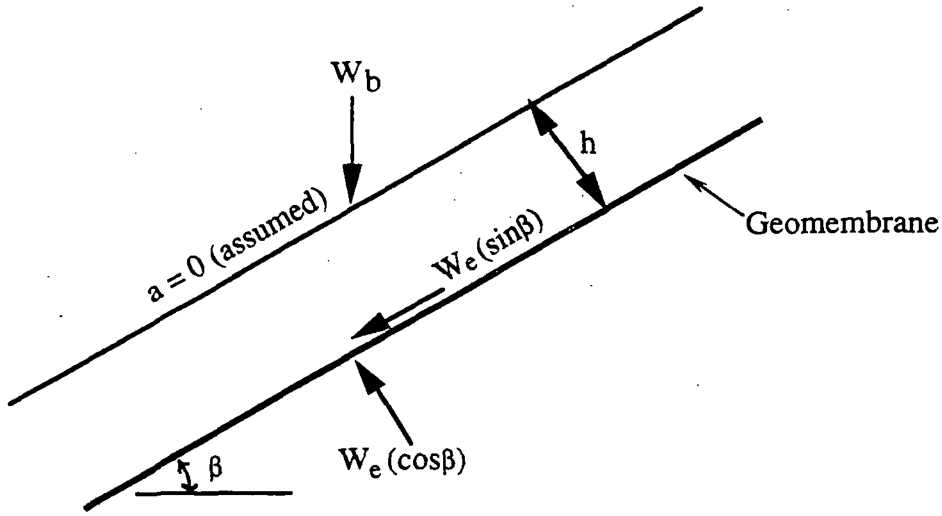
q = $W_b/(2 \times w \times b)$

W_b = actual weight of equipment (e.g., a bulldozer)

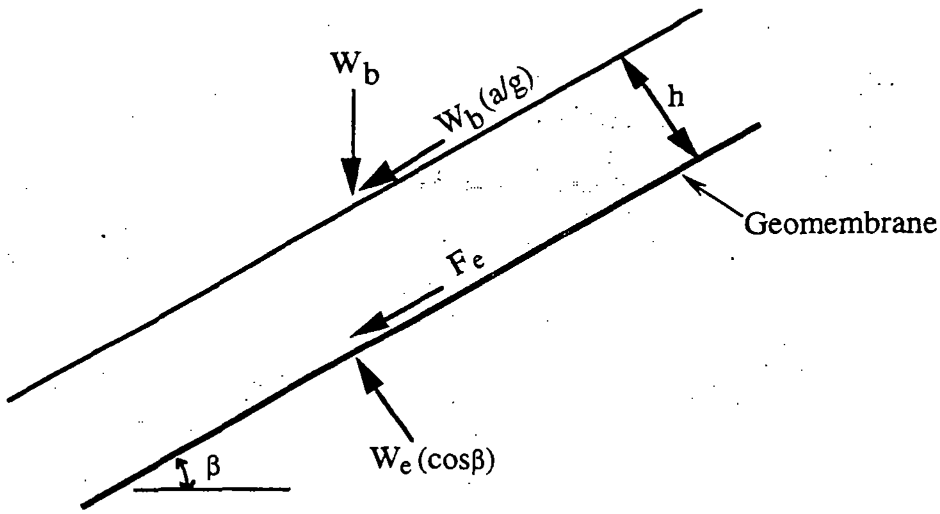
w = length of equipment track

b = width of equipment track

I = influence factor at the geomembrane interface, see Figure 7



(a) Equipment moving up slope



(b) equipment moving down slope

Figure 6 - Additional (to gravitational forces) limit equilibrium forces due to construction equipment moving on cover soil (see Figure 3 for gravitational soil forces which remain the same)

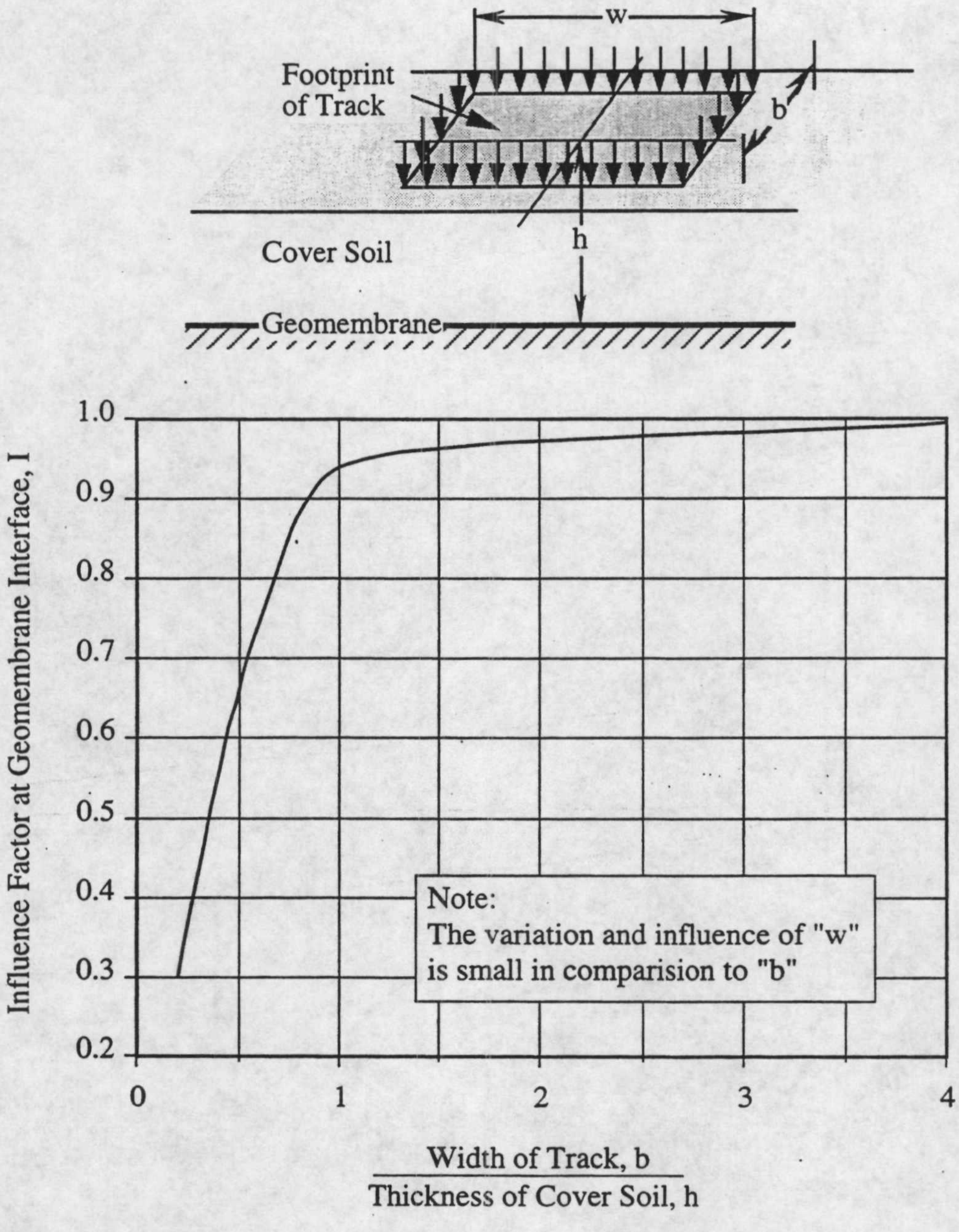


Figure 7 - Values of influence factor, "I", for use in Equations (16) and (17) to dissipate surface force through the cover soil to the geomembrane interface, after Poulos and Davis (1974)

Upon determining the additional equipment force at the cover soil-to-geomembrane interface, the analysis proceeds as described in section 3.1 for gravitational forces only. In essence, the equipment moving up the slope adds an additional term, W_e, to the W_A-force in equation (3). Note, however, that this involves the generation of a resisting force as well. Thus, the net effect of increasing the driving force as well as the resisting force is somewhat neutralized insofar as the resulting FS-value is concerned.

Using this concept (the same equations used in section 3.1 are used), typical design curves for various FS-values as a function of equivalent ground contact pressures and cover soil thicknesses are given in Figure 8. Note that the curves are developed specifically for the variables stated in the legend. Example problem #2(a) illustrates the use of the curves.

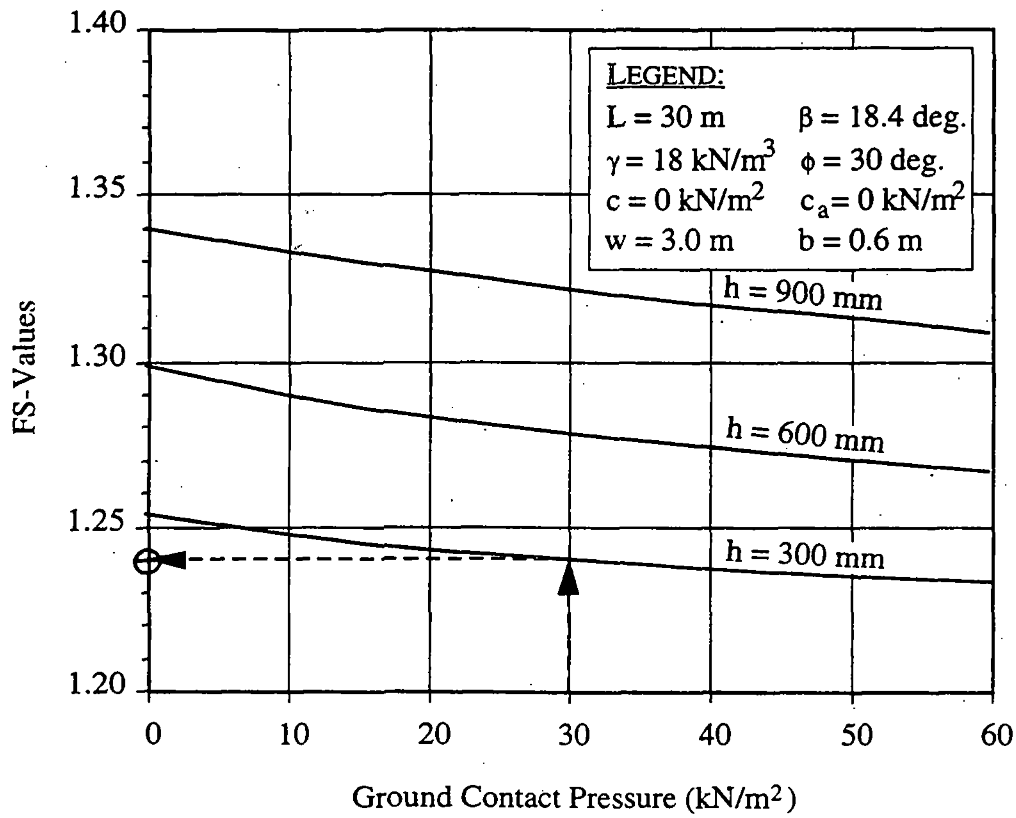


Figure 8 - Design curves for stability of different thickness of cover soil for various construction equipment ground contact pressures

Example 2(a): Given 30 m long slope with uniform cover soil of 300 mm thickness at a unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. It is placed on the slope using a bulldozer moving from the toe of the slope up to the crest. The bulldozer has a ground pressure of 30 kN/m^2 and tracks that are 3.0 m long and 0.6 m wide. The cover soil to geomembrane friction angle is 22 deg. and zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg.

Solution: This problem follows example #1 exactly except for the addition of the bulldozer moving up the slope. Using the design curves of Figure 8 (which were developed for the exact conditions of the example problem), the resulting FS = 1.24.

Comment: While the resulting FS-value is low, the result is best assessed by comparing it to example #1, i.e., the same problem except without the bulldozer. It is seen that the FS-value has only decreased from 1.25 to 1.24. Thus, in general, a low ground contact pressure bulldozer placing cover soil up the slope does not significantly decrease the factor-of-safety .

For the second case of a bulldozer pushing cover soil down from the crest of the slope to the toe, the analysis uses the force diagram of Figure 6(b). While the weight of the equipment is treated as just described, an additional force due to acceleration (or deceleration) of the equipment must be added to the analysis. This analysis again uses a specific piece of construction equipment operated in a specific manner. It produces a force parallel to the slope equivalent to W_b (a/g), where W_b = the weight of the bulldozer, a = acceleration of the bulldozer and g = acceleration due to gravity. Its magnitude is equipment operator dependent and related to both the equipment speed and time to reach such a speed, see Figure 9.

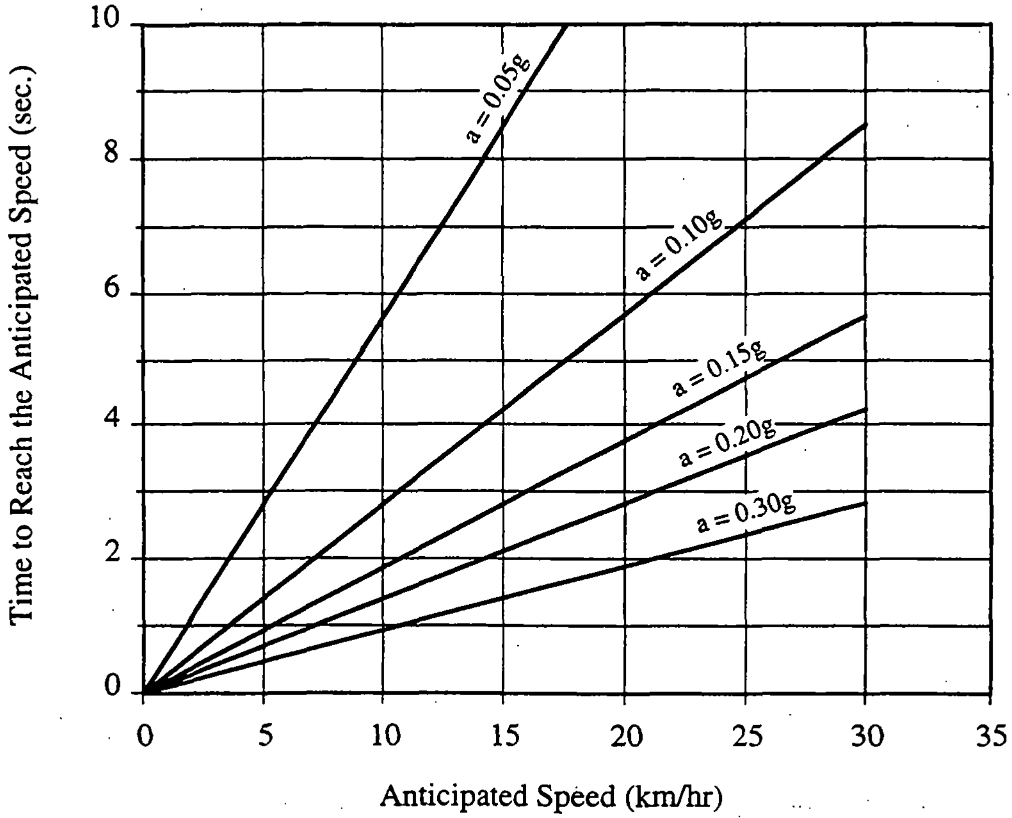


Figure 9 - Graphic relationship of construction equipment speed and rise time to obtain equipment acceleration

The acceleration of the bulldozer, coupled with an influence factor "I" (from Figure 7), results in the dynamic force per unit width at the cover soil to geomembrane interface, "F_e". The relationship is as follows:

$$F_e = W_e\left(\frac{a}{g}\right) \tag{17}$$

where

F_e = dynamic force per unit width parallel to the slope at the geomembrane interface,

W_e = equivalent equipment (bulldozer) force per unit width at geomembrane interface, recall Equation (16).

β = soil slope angle beneath geomembrane

a = acceleration of the bulldozer

g = acceleration due to gravity

I = influence factor at the geomembrane interface, see Figure 7

Using these concepts, the new force parallel to the cover soil surface is dissipated through the thickness of the cover sand to the interface of the geomembrane. Again, a Boussinesq analysis is used, see Poulos and Davis (1974). The expression for finding the FS-value can now be derived as follows:

Considering the active wedge, and balancing the forces in the direction parallel to the slope, the following formulation results:

Doesn't match Section 5 text

$$E_A + \frac{(N_e + N_A)\tan\delta + C_a}{FS} = (W_A + W_e)\sin\beta + F_e \tag{18}$$

Hand Calc

where Ne = effective equipment force normal to the failure plane of the active wedge

$$= (W_e \cos\beta) \qquad N_e = I^2 \tag{19}$$

Note that all ~~the other~~ symbols have been previously defined.

The interwedge force acting on the active wedge can now be expressed as:

$$E_A = \frac{(FS)[(W_A + W_e)\sin\beta + F_e] - [(N_e + N_A)\tan\delta + C_a]}{FS} \tag{20}$$

The passive wedge can be treated in a similar manner and the following formulation of the interwedge force acting on the passive wedge results:

$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi} \tag{21}$$

By setting $E_A = E_P$, the following equation can be arranged in the form of equation (13) in which the "a", "b" and "c" terms are defined as follows:

$$a = [(W_A + W_e)\sin\beta + F_e]\cos\beta$$

$$b = -\{[(N_e + N_A)\tan\delta + C_a]\cos\beta + [(W_A + W_e)\sin\beta + F_e]\sin\beta\tan\phi + (C + W_P\tan\phi)\}$$

$$c = [(N_e + N_A)\tan\delta + C_a]\sin\beta\tan\phi \tag{22}$$

Finally, the resulting FS-value can be obtained using equation (15).

Using these concepts, typical design curves for various FS-values as a function of equipment ground contact pressure and equipment acceleration can be developed, see Figure 10. Note that the curves are developed specifically for the variables stated in the legend. Example problem #2(b) illustrates the use of the curves.

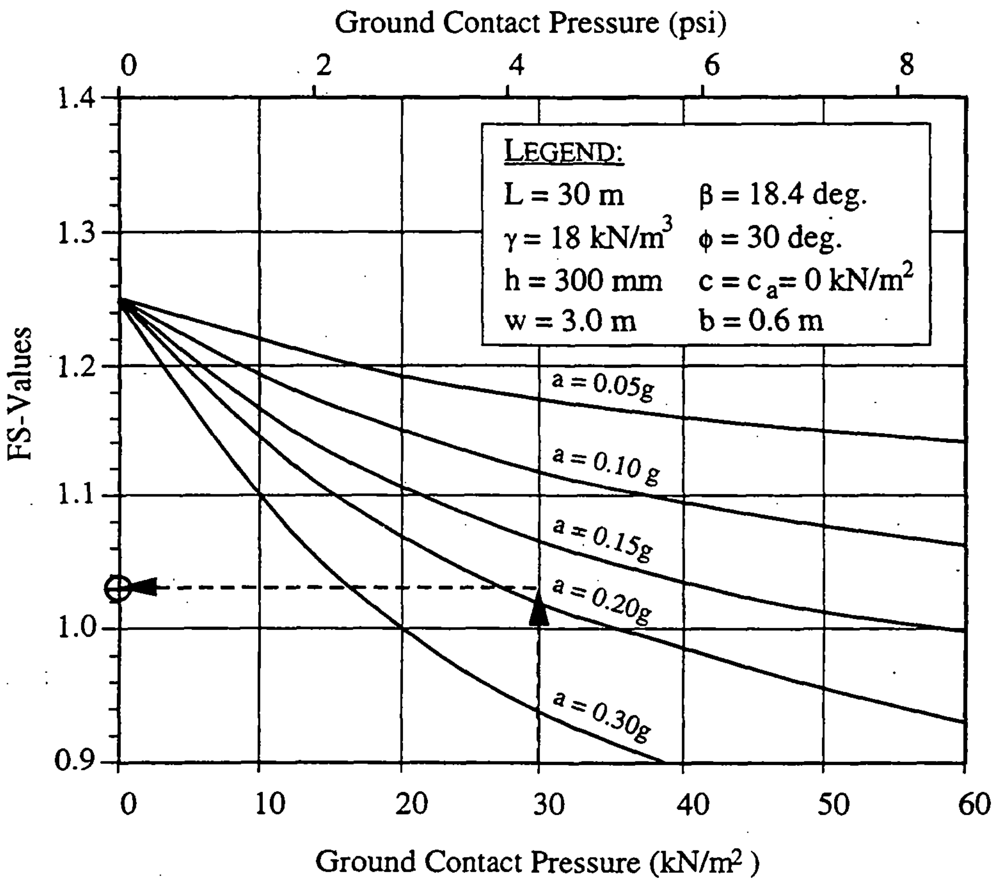


Figure 10 - Design curves for stability of different construction equipment ground contact pressure for various equipment accelerations

Example 2(b): Given a 30 m long slope with uniform cover soil of 300 mm thickness at a unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. It is placed on the slope using a bulldozer moving from the crest of the slope down to the toe. The bulldozer has a ground contact pressure of 30 kN/m^2 and tracks that are 3.0 m long and 0.6 m wide. The estimated equipment speed is 20 km/hr and the time to reach this speed is 3.0 sec.

The cover soil to geomembrane friction angle is 22 deg. and zero adhesion. What is the FS-value at a slope angle of 3(H)-1(V), i.e., 18.4 deg.

Solution: Using the design curves of Figures 9 and 10 (which were developed for the exact conditions of the example problem), we obtain the following:

- From Figure 9 at 20 km/hr and 3.0 sec. the bulldozer's acceleration is 0.19g.
- From Figure 10 (which is developed for a cover soil thickness of 300 mm) at a value of a = 0.19 g and q = 30 kPa, the resulting FS = 1.03.

Comment: This problem solution can now be compared to the previous two examples:

Ex. 1:	cover soil alone with no bulldozer	FS = 1.25
Ex. 2(a):	cover soil plus bulldozer moving up slope	FS = 1.24
Ex. 2(b):	cover soil plus bulldozer moving down slope	FS = 1.03

The inherent danger of a bulldozer moving down the slope is readily apparent. Note, that the same result comes about by the bulldozer decelerating instead of accelerating. The sharp breaking action of the bulldozer is arguable the more severe condition due to the extremely short times involved when stopping forward motion. Clearly, only in unavoidable situations should the cover soil placement equipment be allowed to work down the slope. If it is unavoidable, an analysis should be made of the specific stability situation and the construction specifications should reflect the exact conditions made in the analysis. At the minimum, the ground contact pressure of the equipment should be stated along with suggested operator control of the cover soil placement operations. Truck traffic on the slopes can also give as high, or even higher, stresses and should be avoided in all circumstances.

3.3 Slopes with Tapered Thickness Cover Soils

One method available to the designer to increase the FS-value of a slope is to taper the cover soil thickness from thick at the toe, to thin at the crest, see Figure 11. The FS-value will increase in approximate proportion to the thickness of soil at the toe. The analysis for tapered cover soils includes the design assumptions of a tension crack at the top of the slope, the upper surface of the cover soil tapered at a constant angle "ω", and the earth pressure forces on the respective wedges oriented at the average of the surface and slope angles, i.e., the E-forces are at an angle of (ω+β)/2. The procedure follows that of the uniform cover soil thickness analysis. Again, the resulting equation is not an explicit solution for the FS, and must be solved indirectly.

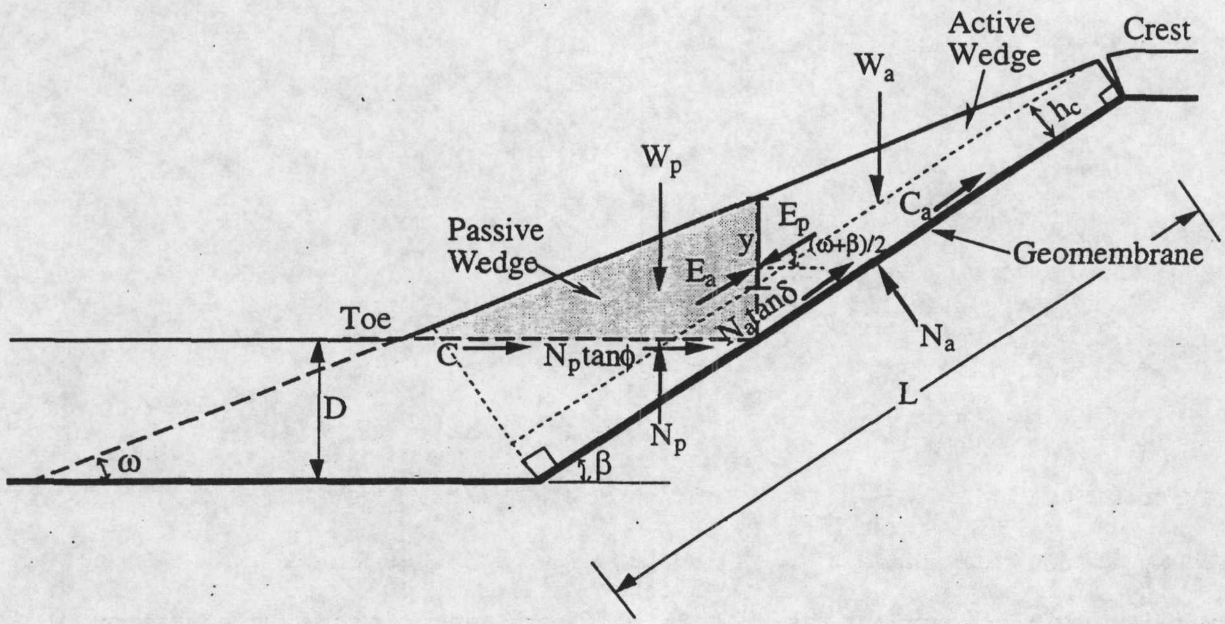


Figure 11 -Limit equilibrium forces involved in a finite length slope analysis with tapered thickness cover soil from toe to crest

All symbols used in Figure 11 were previously defined (see section 3.1) except the following:

D = thickness of cover soil at bottom of the landfill, measured vertically

h_c = thickness of cover soil at crest of the slope, measured perpendicular to the slope

y = see Figure 11 $= (L - \frac{D}{\sin\beta} - h_c \tan\beta)(\sin\beta - \cos\beta \tan\omega)$

ω = finished cover soil slope angle, note that ω < β

The expression for determining the FS-value can be derived as follows:

Considering the active wedge,

$$W_A = \gamma\left[\left(L - \frac{D}{\sin\beta} - h_c \tan\beta\right)\left(\frac{y\cos\beta}{2} + h_c\right) + \frac{h_c^{\,2}\tan\beta}{2}\right] \qquad (23)$$

$$N_A = W_A \cos\beta \tag{24}$$

$$C_a = c_a\left(L - \frac{D}{\sin\beta}\right) \tag{25}$$

By balancing the forces in the vertical direction, the following formulation results:

$$E_A \sin(\frac{\omega+\beta}{2}) = W_A - N_A \cos\beta - \frac{N_A \tan\delta + C_a}{FS}(\sin\beta) \tag{26}$$

Hence the interwedge force acting on the active wedge is:

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin(\frac{\omega+\beta}{2})(FS)} \tag{27}$$

The passive wedge can be considered in a similar manner:

$$W_P = \frac{\gamma}{2\tan\omega}\left[\left(L - \frac{D}{\sin\beta} - h_c\tan\beta\right)(\sin\beta - \cos\beta\tan\omega) + \frac{h_c}{\cos\beta}\right]^2 \tag{28}$$

$$N_p = W_P + E_P \sin\left(\frac{\omega+\beta}{2}\right) \tag{29}$$

$$C = \frac{\gamma}{\tan\omega}\left[\left(L - \frac{D}{\sin\beta} - h_c\tan\beta\right)(\sin\beta - \cos\beta\tan\omega) + \frac{h_c}{\cos\beta}\right] \tag{30}$$

By balancing the forces in the horizontal direction, the following formulation results:

$$E_P \cos\left(\frac{\omega+\beta}{2}\right) = \frac{C + N_P\tan\phi}{FS} \tag{31}$$

Hence the interwedge force acting on the passive wedge is:

$$E_P = \frac{C + W_P\tan\phi}{\cos\left(\frac{\omega+\beta}{2}\right)(FS) - \sin\left(\frac{\omega+\beta}{2}\right)\tan\phi} \tag{32}$$

Again, by setting $E_A = E_P$, the following equation can be arranged in the form of $ax^2+bx+c=0$ which in our case is

$$a(FS)^2 + b(FS) + c = 0 \qquad (13)$$

where

$$a = (W_A - N_A \cos\beta)\cos\left(\frac{\omega+\beta}{2}\right)$$

$$b = -[(W_A - N_A \cos\beta)\sin\left(\frac{\omega+\beta}{2}\right)\tan\phi + (N_A \tan\delta + C_a)\sin\beta\cos\left(\frac{\omega+\beta}{2}\right)$$
$$+ \sin\left(\frac{\omega+\beta}{2}\right)(C + W_P \tan\phi)]$$

$$c = (N_A \tan\delta + C_a)\sin\beta\sin\left(\frac{\omega+\beta}{2}\right)\tan\phi \qquad (33)$$

Again, the resulting FS-value can then be obtained using equation (15).

To illustrate the use of the above developed equations, the design curves of Figure 12 are offered. They show that the FS-value increases in proportion to greater cover soil thicknesses at the toe of the slope with respect to the thickness at the crest. This is evidenced by a shallower finished cover soil slope angle than that of the slope angle of the geomembrane and the soil beneath, i.e., the value of "ω" being less than "β". Note that the curves are developed specifically for the variables stated in the legend. Example problem #3 illustrates the use of the curves.

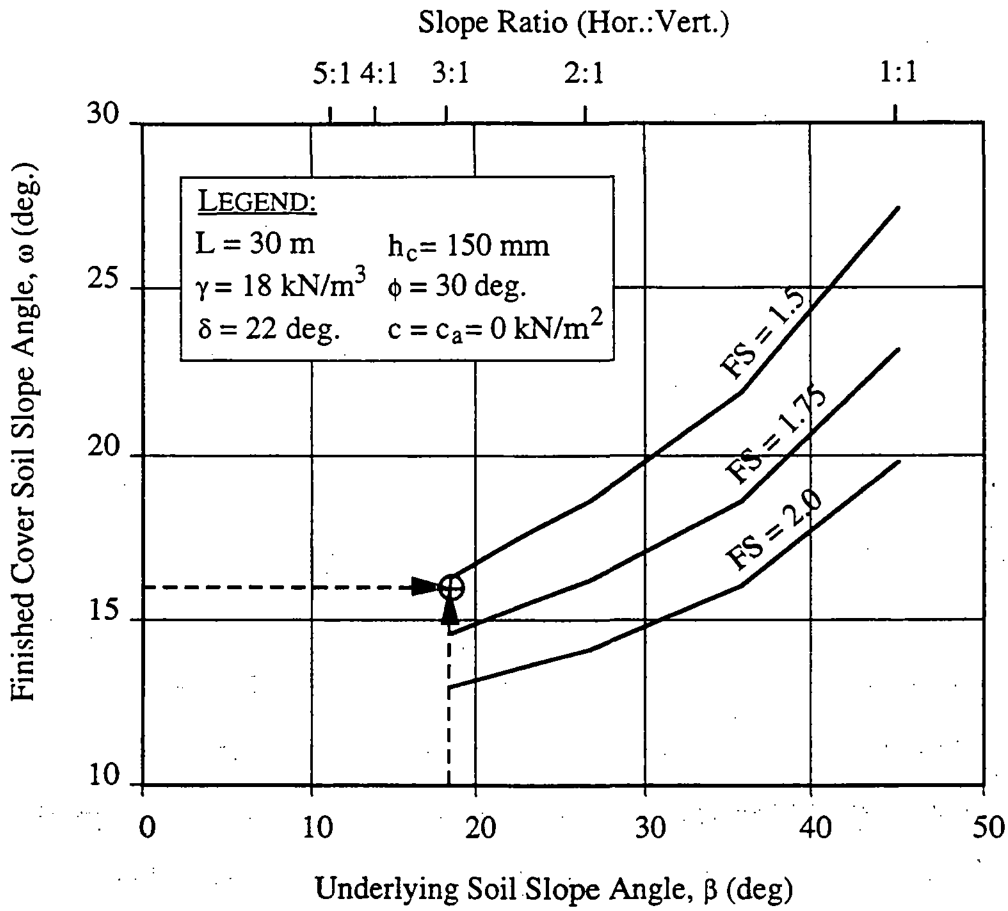


Figure 12 - Design curves for FS-values of tapered cover soil thickness

Example 3: Given a 30 m long slope with a tapered thickness cover soil of 150 mm at the crest extending at an angle "ω" of 16 deg. to the intersection of the cover soil at the toe. The unit weight of the cover soil is 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The interface friction angle with the underlying geomembrane is 22 deg. and zero adhesion. What is the FS-value at an underlying soil slope angle "β" of 3(H)-to-1(V), i.e., 18.4 deg.?

Solution: Using the design curves of Figure 12 (which were developed for the exact conditions of the example problem), the resulting FS = 1.57.

Comment: The result of this problem (with tapered thickness cover soil) is FS = 1.57, versus example #1 (with a uniform thickness cover soil) which was FS = 1.25. Thus the increase in FS-value is 24%. Note, however, that at ω = 16 deg. the thickness of the cover soil normal to the slope at the toe is approximately 1.4 m. Thus the increase in cover soil *volume* used over example #1 is from 8.9 to 24.1 m^3/m (≈ 170%) and the increase in necessary *toe distance* is from 1.0 to 4.8 m (≈ 380%). The trade-offs between these issues should be considered when using the strategy of tapered cover soil thickness to increase the FS-value of a particular cover soil slope.

3.4 Veneer Reinforcement of Cover Soil Slopes

A fundamentally different way of increasing a given slope's factor-of-safety is to reinforce it with a geosynthetic material. Such reinforcement can be either *intentional* or *non-intentional.* By intentional, we mean to include a geogrid or high strength geotextile within the cover soil to purposely reinforce the system against instability, see Figure 13. Depending on the type and amount of reinforcement, the majority, or even all, of the gravitational stresses can be supported resulting in a major increase in the FS-value. By non-intentional, we refer to multi-component liner systems where a low shear strength interface is located beneath an overlying geosynthetic(s). In this case, the overlying geosynthetic(s) is inadvertently acting as veneer reinforcement to the system. In some cases, the designer may not realize that such geosynthetic(s) are being stressed in an identical manner as a geogrid or high strength geotextile, but they are. The situation where a relatively low strength protection geotextile is placed over a geomembrane and beneath the cover soil is a case in point. Intentional, or non-intentional, the stability analysis is identical. The difference is that the geogrids and/or high strength geotextiles give a major increase in the FS-value, while a protection geotextile (or other lower strength geosynthetics) only nominally increases the FS-value.

Seen in Figure 13 is that the analysis follows section 3.1, but a force from the reinforcement "T", acting parallel to the slope, provides additional stability. This force "T", acts only within the active wedge. By taking free body force diagrams of the active and passive wedges, the following formulation for the factor-of-safety results. All symbols used in Figure 13 were previously defined (see section 3.1) except the following:

$T = T_{allow}$

= the allowable (long-term) strength of the geosynthetic reinforcement inclusion

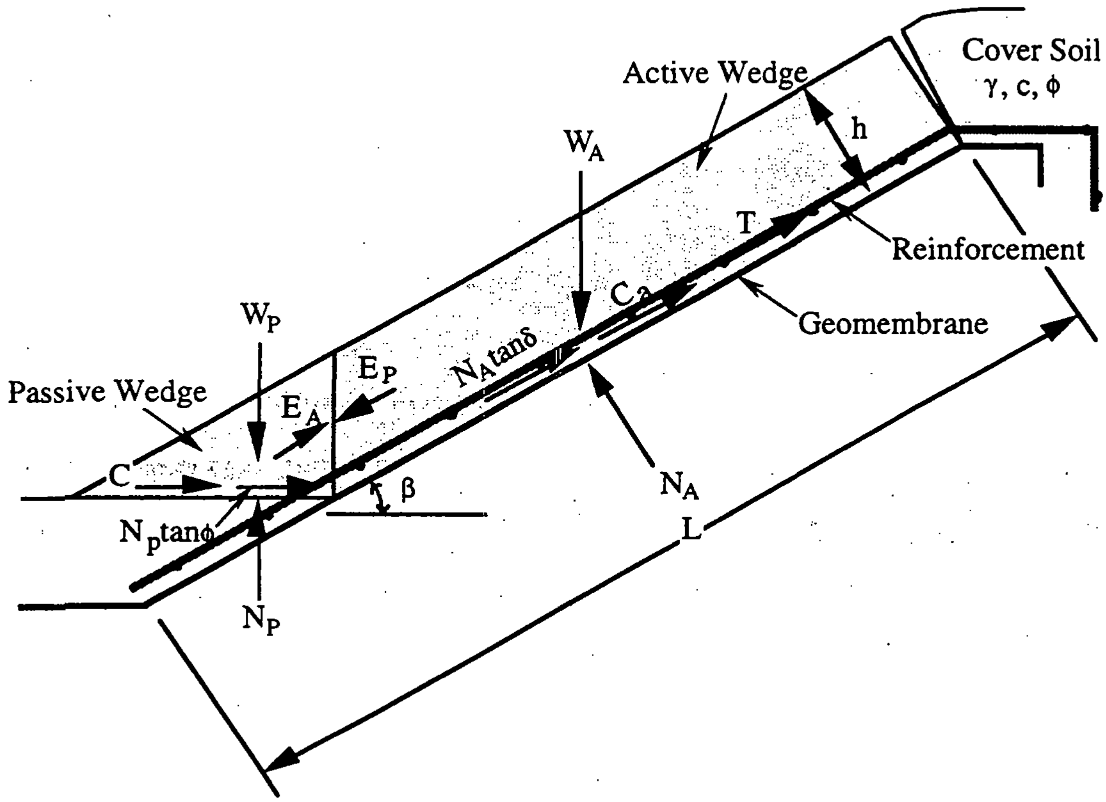


Figure 13- Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil including the use of veneer reinforcement

Considering the active wedge, by balancing the forces in the vertical direction, the following formulation results:

$$E_A \sin\beta = W_A - N_A \cos\beta - \left(\frac{N_A \tan\delta + C_a}{FS} + T\right)\sin\beta \tag{34}$$

Hence the interwedge force acting on the active wedge is:

$$E_A = \frac{(FS)(W_A - N_A \cos\beta - T\sin\beta) - (N_A \tan\delta + C_a)\sin\beta}{\sin\beta(FS)} \tag{35}$$

Again, by setting $E_A = E_P$, (see equation (12) for the expression of Ep), the following equation can be arranged in the form of equation (13) in which the "a", "b" and "c" terms are defined as follows:

$$a = \left(W_A - N_A \cos\beta - T\sin\beta\right)\cos\beta$$

$$b = -[(W_A - N_A \cos\beta - T\sin\beta)\sin\beta\tan\phi + (N_A \tan\delta + C_a)\sin\beta\cos\beta$$
$$+ \sin\beta(C + W_P \tan\phi)]$$

$$c = (N_A \tan\delta + C_a)\sin^2\beta\tan\phi \tag{36}$$

Again, the resulting FS-value can be obtained using equation (15).

The value of allowable tensile strength is necessary for the long-term stability of the cover soil. In order to obtain an ultimate, or as-manufactured, tensile strength of the geogrid or high strength geotextile, the value of T_{allow} must be *increased* for site specific conditions via partial factors-of-safety. Such values as installation damage, creep and long-term degradation are generally considered, see equation 37 after Koerner (1994). Note that if seams are involved in the reinforcement, a partial factor-of-safety should be added accordingly.

$$T_{allow} = T_{ult}\left(\frac{1}{FS_{ID} \times FS_{CR} \times FS_{CBD}}\right) \tag{37}$$

where

T_{allow}	= allowable value of reinforcement strength
T_{ult}	= ultimate (as-manufactured) value of reinforcement strength
FS_{ID}	= partial factor-of-safety for installation damage
FS_{CR}	= partial factor-of-safety for creep
FS_{CBD}	= partial factor-of-safety for chemical/biological degradation

To illustrate the use of the above developed equations, the design curves of Figure 14 have been developed. They show the improvement of FS-values with increasing strength of the reinforcement. Note that the curves are developed specifically for the variables stated in the legend. Example problem #4 illustrates the use of the design curves.

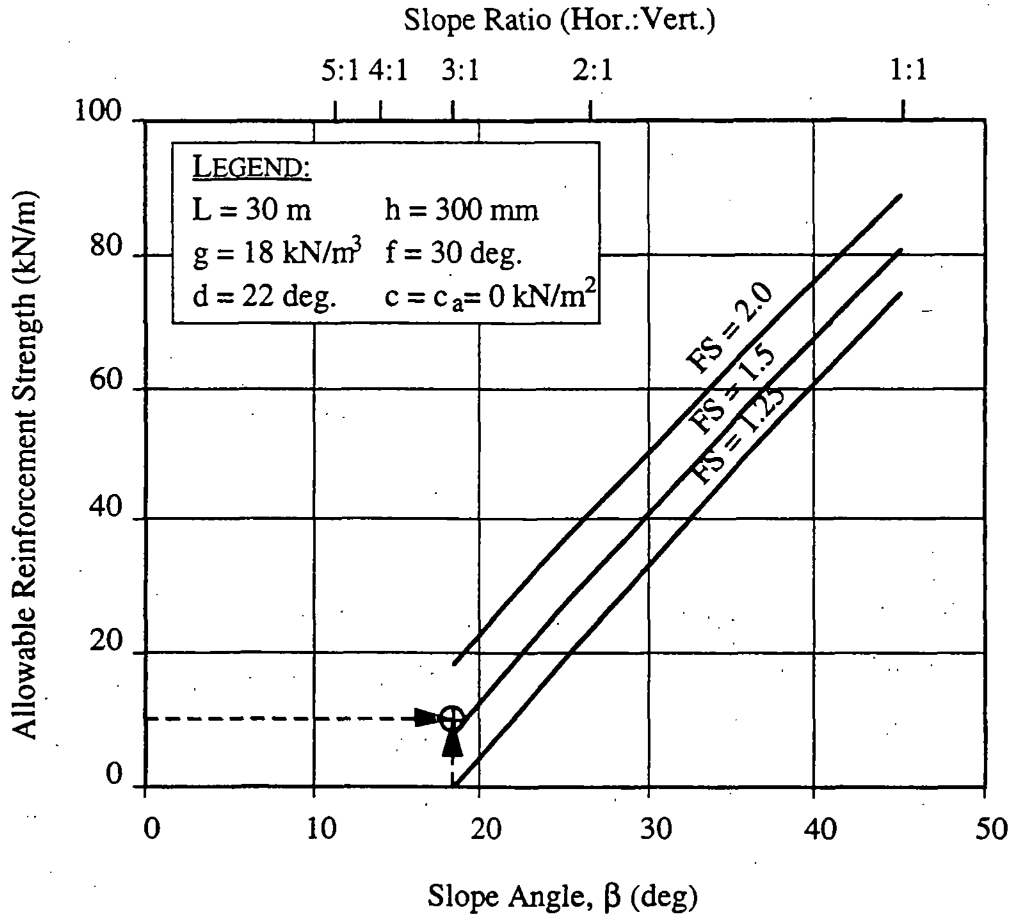


Figure 14- Design curves for FS-values for different slope angles and veneer reinforcement strengths of uniform thickness cohesionless cover soils

Example 4: Given a 30 m long slope with a uniform thickness cover soil of 300 mm and a unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The proposed reinforcement is a geogrid with an allowable wide width tensile strength of 10 kN/m. Thus the partial factors-of-safety in equation (37) have already been included. The geogrid apertures are large enough that the cover soil will strike-through and provide an interface friction angle with the underlying geomembrane of 22 deg. and zero adhesion. What is the FS-value at a side slope angle of 3(H)-to-1(V), i.e., 18.4 deg.?

Solution: Using the design curves of Figure 14 (which were developed for the exact conditions of the example problem), the resulting FS = 1.57.

Comments: Note that the use of T_{allow} = 10 kN/m in the analysis will require a significantly higher T_{ult} value of the geogrid per equation 37. For example, if the total partial factors-of-safety in equation 37 was 4.0, the ultimate (as-manufactured) strength of the geogrid would have to be 40 kN/m. Also, note that this same type of analysis could also be used for high strength geotextile reinforcement. The only difference is that strike-through of the cover soil will not occur. Hence, the interface shear strength of concern is the geotextile to the underlying geomembrane. The analysis follows along the same general lines as presented here. More detail on this latter situation is provided in Appendix "A".

It should be emphasized that the preceding analysis is focused on *intentionally* improving the FS-value by the inclusion of geosynthetic reinforcement. This is provided by geogrids or high strength geotextiles being placed above the upper surface of the low strength interface material. The reinforcement is usually placed directly above the geomembrane or other geosynthetic material.

Interestingly, some amount of veneer reinforcement is often *non-intentionally* provided by a protection geotextile placed above a geomembrane lined slope and beneath the cover soil. The geotextile will obviously be highly stressed by the cover soil and if its interface friction strength against the underlying geomembrane is relatively low, it will tend to slide on the underlying geomembrane. When contained in an anchor trench, however, the protection geotextile actually acts as a de-facto reinforcement material. Since its wide width tensile strength is usually low, it does relatively little to improve the slope's factor-of-safety. Furthermore, when it fails or pulls out of its anchor trench, the sliding of the geotextile (and overlying cover soil) on the underlying (and stationary) geomembrane is very abrupt. This same situation occurs deeper into the cross section for many multiple lined slopes, e.g., with GCLs. Appendix "A" treats a number of common situations. Clearly, if veneer reinforcement is to be provided it must be done intentionally with the proper type of geosynthetic and designed accordingly.

4.0 Consideration of Seepage Forces

The previous section presented the general problem of slope stability analysis of final cover soils placed on slope angles of varying degrees. The tacit assumption throughout was that a drainage layer was placed above the barrier layer with adequate capacity to efficiently remove, i.e., transmit, permeating water parallel to the slope and safely away from the cross section. The amount of water to be removed is obviously a site specific situation. Note that in extremely arid areas, drainage may not be required although this is generally the exception.

Unfortunately, adequate drainage of final covers has sometimes not been available and seepage induced slope stability problems have occurred. The following situations have resulted in seepage induced slides:

- Inadequate drainage capacity at the toe of long slopes where seepage quantities accumulate and are at their maximum.
- Fines from quarried stone accumulating at the toe of the slope thereby decreasing the as-constructed permeability over time.
- Fine, cohesionless, cover soil particles migrating through the filter (if one is even present) and the drainage layer, and then accumulating at the toe of the slope thereby decreasing the as-constructed permeability over time.
- Freezing of the drainage layer at the toe of the slope, while the top of the slope thaws, thereby mobilizing seepage forces against the ice wedge at the toe.

If seepage forces of the types described occur, a variation in slope stability design methodology is required. Such analysis is the focus of this subsection. Detailed discussion is given in Soong and Koerner (1996).

Consider a cover soil of uniform thickness placed directly above a geomembrane at a slope angle of "β" as shown Figure 15. Different from previous examples, however, is that within the cover soil exists a saturated soil zone for part of all of the thickness. The saturated boundary is shown as two possibly different phreatic surface orientations. This is because seepage can be built-up in the cover soil in two different ways: a horizontal buildup from the toe upward and a parallel-to-slope buildup outward. These two hypotheses are defined and quantified as a horizontal submergence ratio (HSR) and a parallel submergence ratio (PSR). The dimensional definitions of both ratios are given in Figure 15.

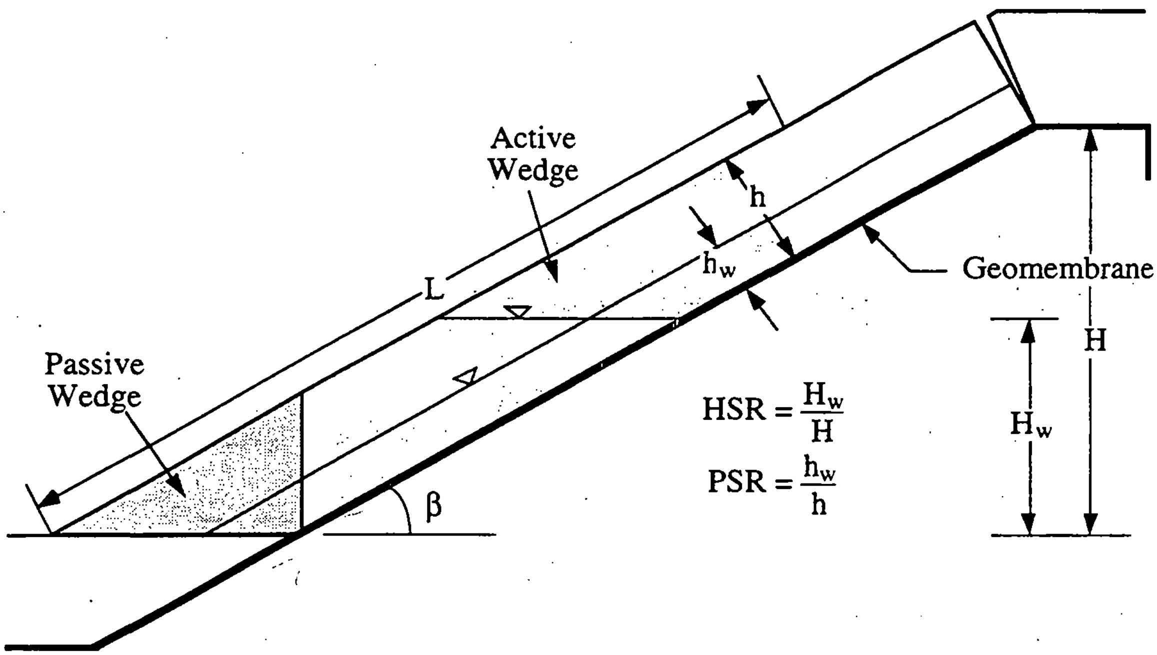


Figure 15- Cross section of a uniform thickness cover soil on a geomembrane illustrating different submergence assumptions and related definitions, Soong and Koerner (1996)

When analyzing the stability of slopes using the limit equilibrium method, free body diagrams of the passive and active wedges are taken with the appropriate forces (now including pore water pressures) being applied. Note that the two interwedge forces, namely, E_a and E_p, are also shown in Figure 15. The formulation for the resulting factor-of-safety, for horizontal seepage buildup and then for parallel-to-slope seepage buildup, follows.

The Case of Horizontal Seepage Buildup. Figure 16 shows the free body diagram of both the active and the passive wedges assuming horizontal seepage buildup.

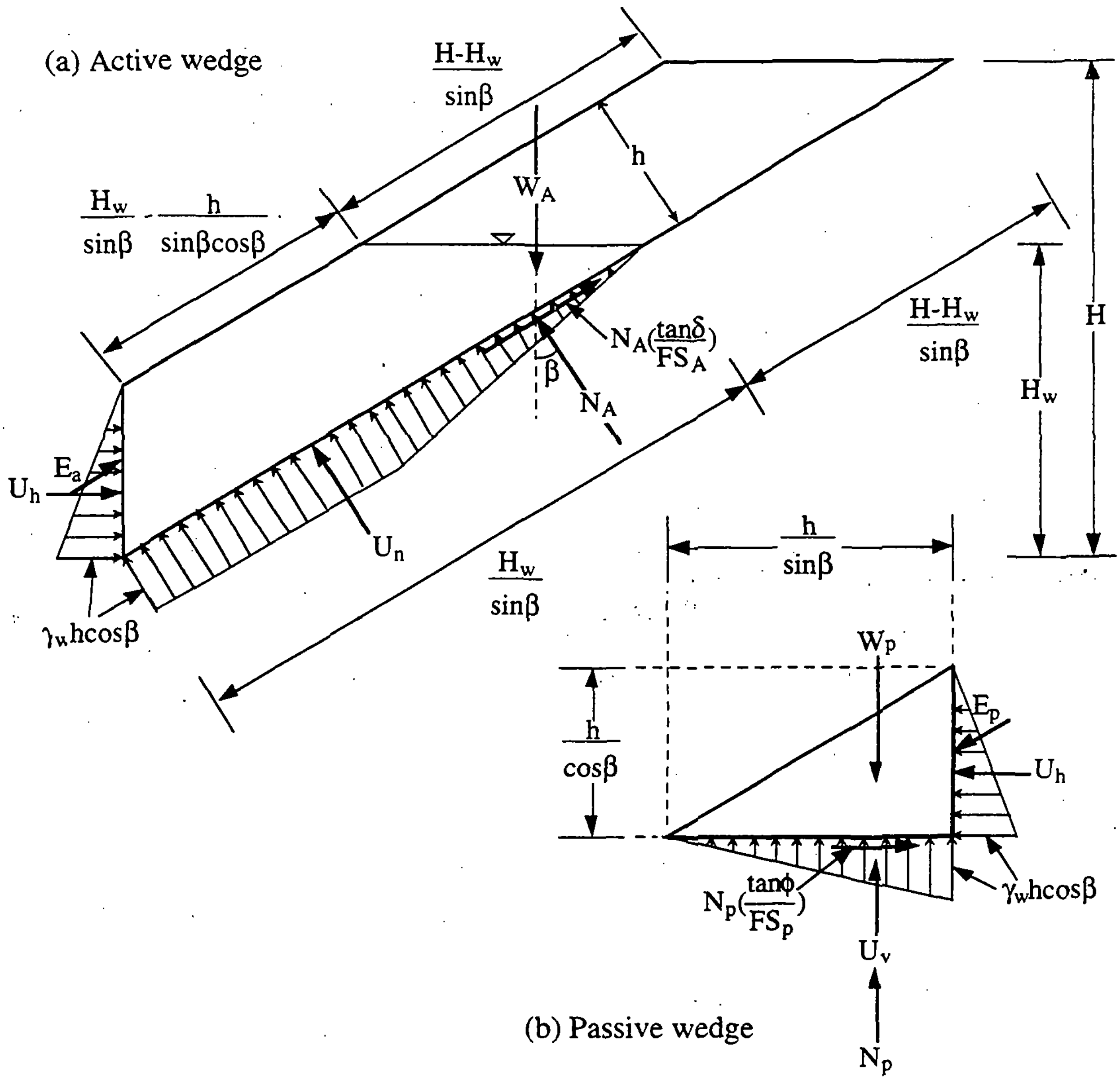


Figure 16 - Limit equilibrium forces involved in a finite length slope of uniform cover soil with horizontal seepage buildup, Soong and Koerner (1996)

All symbols used in Figure 16 were previously defined except the following:

$\gamma_{sat'd}$ = saturated unit weight of the cover soil

γ_{dry} = dry unit weight of the cover soil

γ_w = unit weight of water

H = vertical height of the slope measured from the toe

H_w = vertical height of the free water surface measured from the toe

U_h = resultant of the pore pressures acting on the interwedge surfaces

U_n = resultant of the pore pressures acting perpendicular to the slope

U_v = resultant of the vertical pore pressures acting on the passive wedge

The expression for finding the factor-of-safety can be derived as follows:

Considering the active wedge,

$$W_A = \left(\frac{\gamma_{sat'd}(h)(2H_w\cos\beta - h)}{\sin 2\beta}\right) + \left(\frac{\gamma_{dry}(h)(H - H_w)}{\sin\beta}\right) \tag{38}$$

$$U_n = \frac{\gamma_w(h)(\cos\beta)(2H_w\cos\beta - h)}{\sin 2\beta} \tag{39}$$

$$U_h = \frac{\gamma_w h^2}{2} \tag{40}$$

$$N_A = W_A(\cos\beta) + U_h(\sin\beta) - U_n \tag{41}$$

The interwedge force acting on the active wedge can then be expressed as:

$$E_A = W_A\sin\beta - U_h\cos\beta - \frac{N_A\tan\delta}{FS} \tag{42}$$

The passive wedge can be considered in a similar manner and the following expressions result:

$$W_P = \frac{\gamma_{sat'd}h^2}{\sin 2\beta} \tag{43}$$

$$U_V = U_h\cot\beta \tag{44}$$

The interwedge force acting on the passive wedge can then be expressed as:

$$E_P = \frac{U_h(FS) - (W_P - U_V)\tan\phi}{\sin\beta\tan\phi - \cos\beta(FS)} \tag{45}$$

Again, by setting $E_A = E_P$, the following equation can be arranged in the form of $ax^2+bx+c=0$ which in this case is

$$a(FS)^2 + b(FS) + c = 0 \tag{13}$$

where

$$a = W_A \sin\beta\cos\beta - U_h \cos^2\beta + U_h$$

$$b = -W_A \sin^2\beta \tan\phi + U_h \sin\beta\cos\beta\tan\phi - \mathrm{N_A}\cos\beta\tan\delta - (W_P - U_V)\tan\phi$$

$$c = N_A \sin\beta\tan\delta\tan\phi \tag{46}$$

Again, the resulting FS-value is obtained using equation (15).

<u>The Case of Parallel-to-Slope Seepage Buildup.</u> Figure 17 shows the free body diagrams of both the active and passive wedges with seepage buildup in the direction parallel to the slope. Identical symbols as defined in the previous case are used here with an additional definition of h_W equal to the height of free water surface measured in the direction perpendicular to the slope.

Note that the general expression of factor-of-safety shown in equation (15) is still valid. However, the a, b and c terms shown in equation (46) have different definitions in this case owing to the new definitions of the following terms:

$$W_A = \frac{\gamma_{dry}(h-h_w)(2H\cos\beta-(h+h_w)) + \gamma_{sat'd}(h_w)(2H\cos\beta-h_w)}{\sin 2\beta} \tag{47}$$

$$U_n = \frac{\gamma_w h_w \cos\beta(2H\cos\beta - h_w)}{\sin 2\beta} \tag{48}$$

$$U_h = \frac{\gamma_w (h_w)^2}{2} \tag{49}$$

$$W_P = \frac{\gamma_{dry}(h^2-h_w^2) + \gamma_{sat'd}(h_w^2)}{\sin 2\beta} \tag{50}$$

In order to illustrate the effect of the above developed equations, the design curves of Figure 18 have been developed. They show the decrease in FS-value with increasing submergence ratio for all values of interface friction. Furthermore, the differences in response curves for the parallel and horizontal submergence ratio assumptions are seen to be very small. Note that the curves are developed specifically for the variables stated in the legend. Example problem #5 illustrates the use of the design curves.

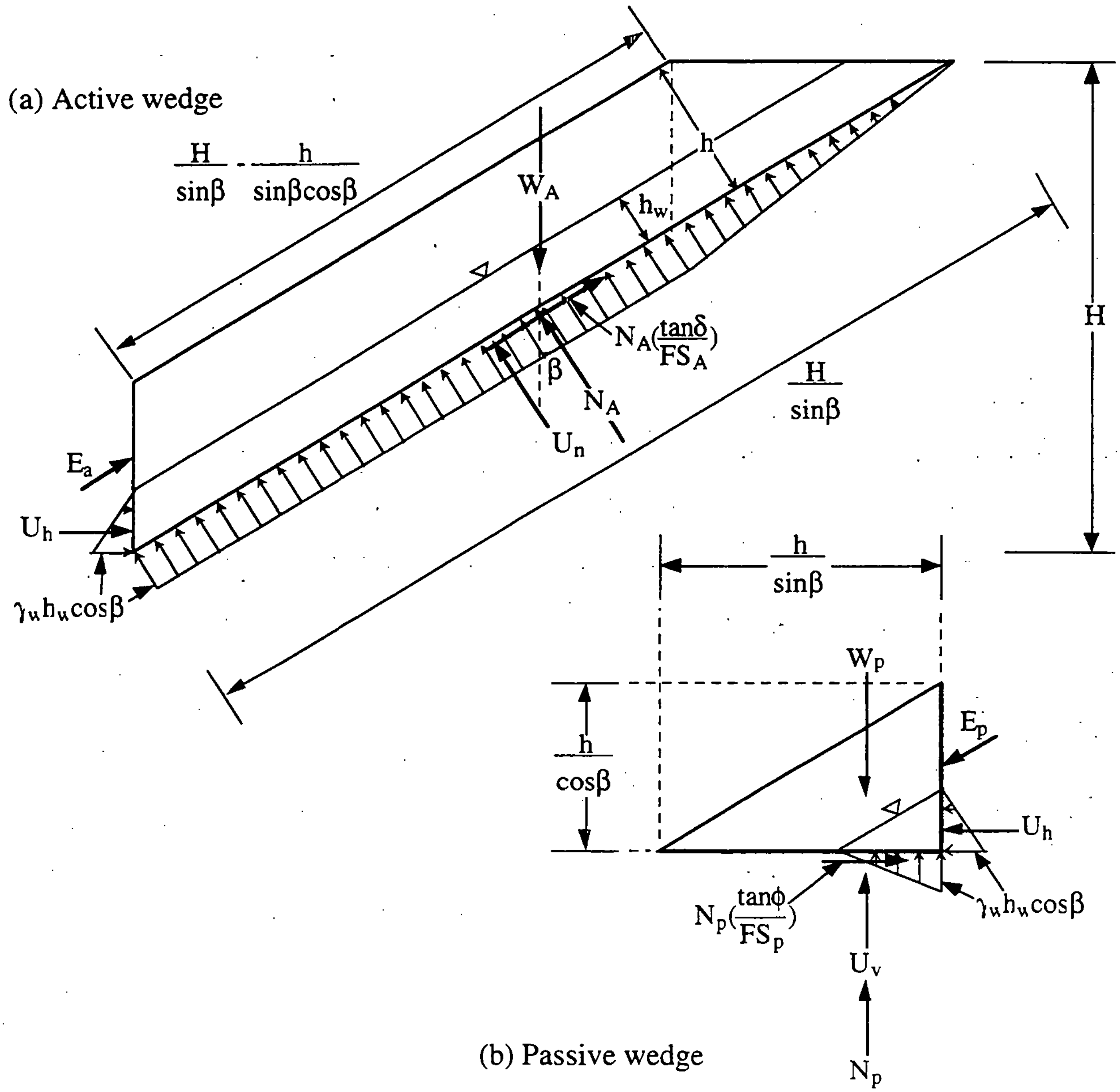


Figure 17 -Limit equilibrium forces involved in a finite length slope of uniform cover soil with parallel-to-slope seepage buildup, Soong and Koerner (1996)

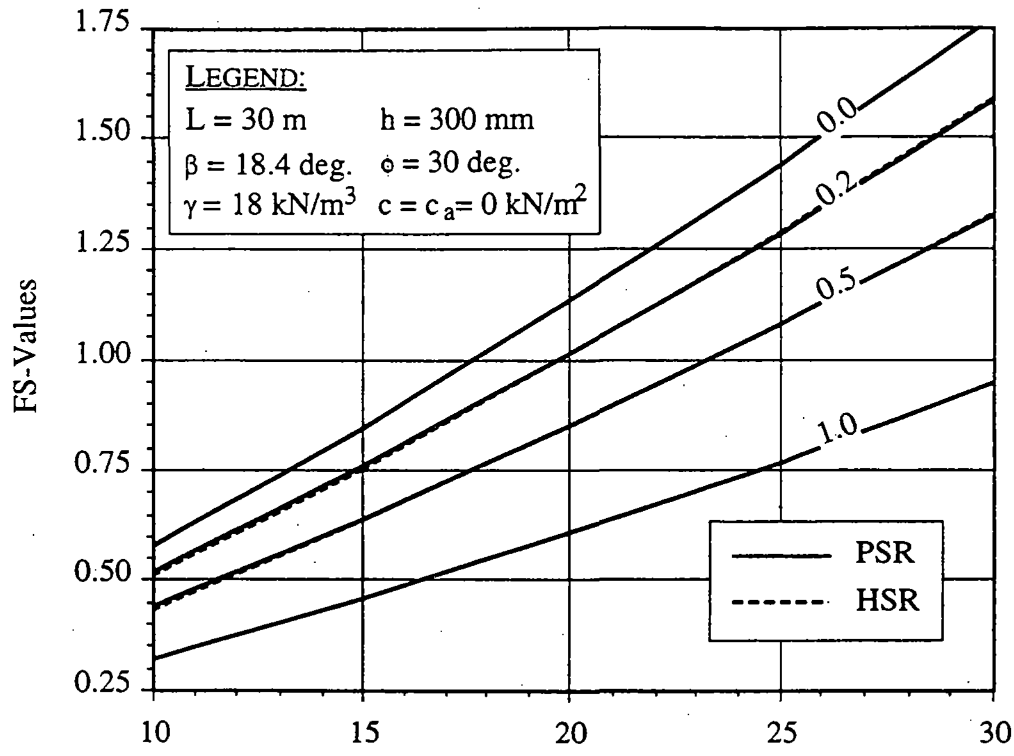


Figure 18 -Design curves for stability of cohesionless, uniform thickness, cover soils for different submergence ratio

<u>Example 5</u>: Given a 30 m long slope with a uniform thickness cover soil of 300 mm at a dry unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The soil becomes saturated through 50% of its thickness, i.e., it is a parallel seepage problem with PSR = 0.5, and its saturated unit weight increases to 21 kN/m^3. Direct shear testing has resulted in an interface friction angle of 22 deg. and zero adhesion. What is the factor-of-safety at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg.

<u>Solution</u>: Using the design curves of Figure 18 (which were developed for the exact conditions of the example problem), the resulting FS = 0.93.

<u>Comment</u>: The seriousness of seepage forces in a slope of this type are immediately obvious. Had the saturation been 100% of the thickness, the FS-value would have been even lower. Furthermore, the result from a horizontal assumption of saturated cover soil with the same

saturation ratio will give almost identically low FS-values. Clearly, the teaching of this example problem is that adequate long-term drainage above the barrier layer in cover soil slopes must be provided to avoid saturation from occurring.

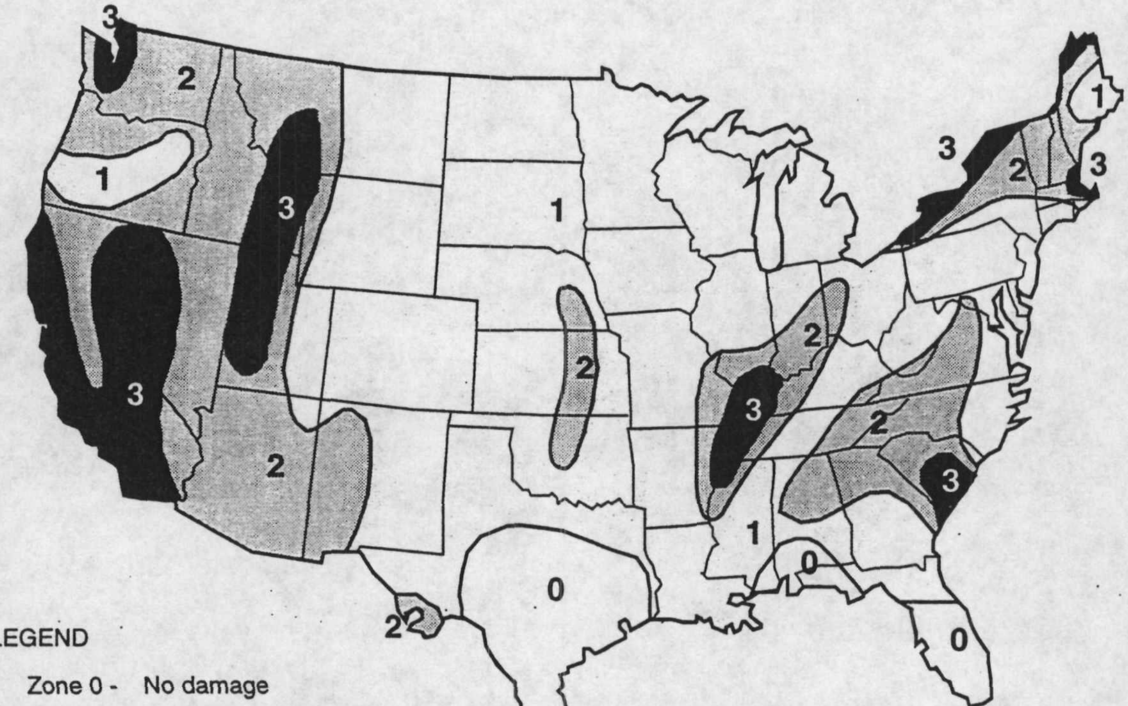
5.0 Consideration of Seismic Forces

In areas of anticipated earthquake activity, the slope stability analysis of a final cover soil over an engineered landfill, abandoned dump or remediated site must consider seismic forces. Subtitle "D" of the U.S. EPA regulations requires such an analysis for sites that have experienced 0.1 g horizontal acceleration, or more, within the past 250 years. This includes zones 3, 2 and most parts of 1 on a seismic zone map as seen in Figure 19. For the continental USA it includes not only the western states, but major sections of the midwest and northeast states, as well. If practiced worldwide, such a criterion would have huge implications.

Seismic analysis of cover soils of the type under consideration in this report is a two-part process.

- The calculation of a FS-value using a pseudo-static analysis via the addition of a horizontal force acting at the centroid of the cover soil cross section.
- If the FS-value in the above calculation falls below 1.0, a permanent deformation analysis is required. The calculated deformation is then assessed in light of the potential damage to the cover soil section and is either accepted, or the slope requires an appropriate redesign. The redesign is then analyzed until the situation becomes acceptable.

The first part of the analysis is a pseudo-static approach which follows the previous examples except for the addition of a horizontal force at the centroid of the cover soil in proportion to the anticipated seismic activity. Figure 19 gives average seismic coefficients for various zones in the USA. Similar maps are available on a worldwide basis. Note that C_S is nondimensional and is a ratio of the bedrock acceleration to gravitational acceleration. This value of C_S is modified using available computer codes such as "SHAKE", see Schnabel et al (1972), for propagation to the landfill cover as shown in Figure 20. In most cases it will be amplified. Detailed discussion see Seed and Idriss (1982) and Idriss (1990). The analysis then proceeds as follows.



LEGEND

Zone 0 - No damage

Zone 1 - Distant earthquakes with fundamental periods greater than 1.0 second may cause minor damage. Corresponds to intensities V and VI on the Modified Mercalli intensity scale.

Zone 2 - Moderate damage; corresponds to intensity VII on the Modified Mercalli intensity scale.

Zone 3 - Major damage; corresponds to intensity VIII and higher on the Modified Mercalli intensity scale.

Seismic coefficients corresponding to each zone.

Zone	Intensity of Modified Mercalli Scale	Average Seismic Coefficient (C_S)	Remark
0	—	0	No damage
1	V and VI	0.03 to 0.07	Minor damage
2	VII	0.13	Moderate damage
3	VII and higher	0.27	Major damage

Figure 19 -Seismic zone map and related seismic coefficients for continental USA, after Algermissen (1969)

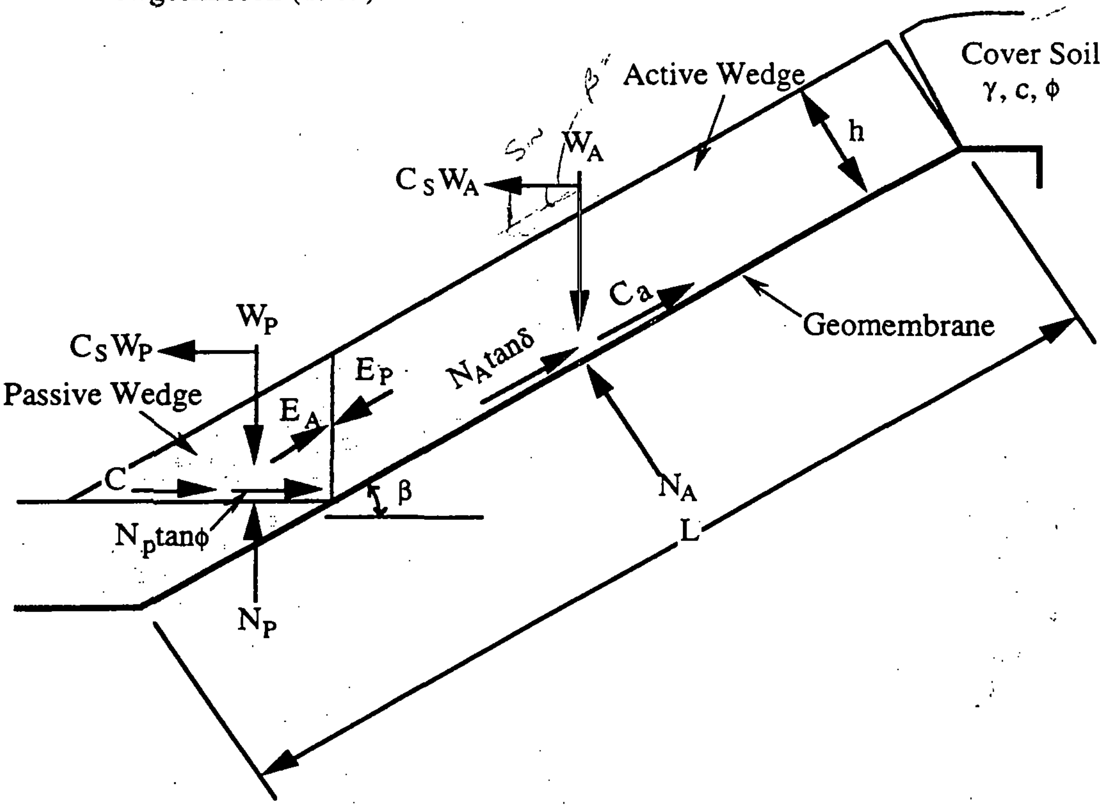


Figure 20 -Limit equilibrium forces involved in pseudo-static analysis using average seismic coefficient

All symbols used in Figure 20 have been previously defined and the expression for finding the factor-of-safety can be derived as follows:

Considering the active wedge, by balancing the forces in the horizontal direction, the following formulation results:

$$E_A \cos\beta + \frac{(N_A \tan\delta + C_a)\cos\beta}{FS} = C_S W_A + N_A \sin\beta \tag{51}$$

Hence the interwedge force acting on the active wedge results:

$$E_a = \frac{(FS)(C_S W_A + N_A \sin\beta) - (N_A \tan\delta + C_a)\cos\beta}{(FS)\cos\beta} \tag{52}$$

The passive wedge can be considered in a similar manner and the following formulation results:

$$E_P \cos\beta + C_S W_P = \frac{C + N_P \tan\phi}{FS} \tag{53}$$

Hence the interwedge force acting on the passive wedge is:

$$E_P = \frac{C + W_P \tan\phi - C_S W_P (FS)}{(FS)\cos\beta - \sin\beta \tan\phi} \tag{54}$$

Again, by setting E_A=E_P, the following equation can be arranged in the form of $ax^2+bx+c=0$ which in our case is:

$$a(FS)^2 + b(FS) + c = 0 \tag{13}$$

where

$$a = (C_S W_A + N_A \sin\beta)\cos\beta + C_S W_P \cos\beta$$

$$b = -[(C_S W_A + N_A \sin\beta)\sin\beta \tan\phi + (N_A \tan\delta + C_a)\cos^2\beta$$
$$+ (C + W_P \tan\phi)\cos\beta]$$

$$c = (N_A \tan\delta + C_a)\cos\beta \sin\beta \tan\phi \tag{55}$$

The resulting FS-value is then obtained from the following equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \tag{15}$$

Using these concepts, a typical design curve for various FS-values as a function of seismic coefficient can be developed, see Figure 21. Note that the curve is developed specifically for the variables stated in the legend. Example problem 6(a) illustrates the use of the curve.

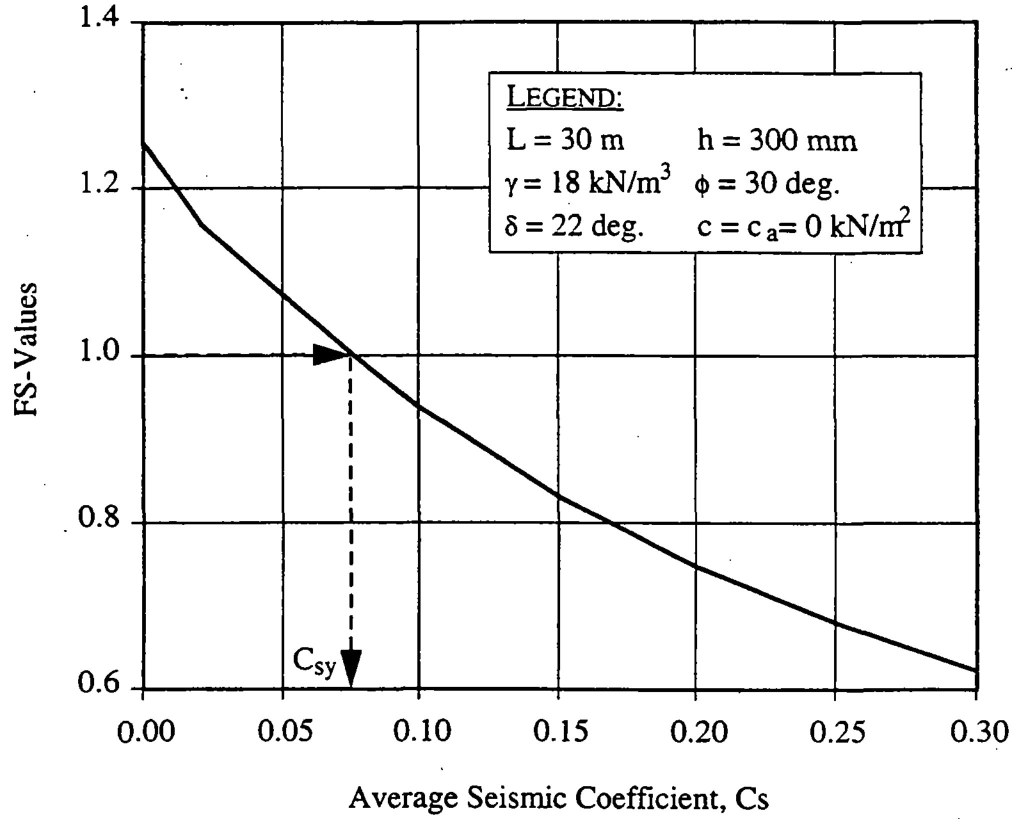


Figure 21 - Design curves for a uniformly thick cover soil pseudo-static seismic analysis with varying average seismic coefficients

Example 6(a): Given a 30 m long slope with uniform thickness cover soil of 300 mm at a unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The cover soil is on a geomembrane as shown in Figure 20. Direct shear testing has resulted in an interface friction angle of 22 deg. and zero adhesion. The slope angle is 3(H)-to-1(V), i.e., 18.4 deg. A design earthquake appropriately transferred to the site results in an average seismic coefficient of 0.10. What is the FS-value?

Solution: Using the design curve of Figure 21 (which was developed for the exact conditions of the example problem), the resulting FS = 0.94.

Comment: Had the above FS-value been greater than 1.0, the analysis would be complete. The assumption being that cover soil stability can withstand the short-term excitation of an earthquake and still not fail. However, since the value is less than 1.0, a second part of the analysis is required.

The second part of the analysis is based on calculating the estimated deformation of the lowest shear strength interface in the cross section under consideration. The deformation is then assessed in light of the potential damage that may be imposed on the final cover system.

To begin the permanent deformation analysis, a yield acceleration, "C_{sy}", is obtained from a pseudo-static analysis under an assumed FS = 1.0. Figure 21 illustrates this procedure for the assumptions stated in the legend. It results in a value of $C_{sy} = 0.075$. Coupling this value with the spectrum obtained for the actual site location and cross section, results in a comparison as shown in Figure 22(a). If the earthquake spectrum never exceeds the value of C_{sy}, there is no anticipated permanent deformation. However, whenever any part of the spectrum exceeds the value of C_{sy}, permanent deformation is expected. By double integration of the acceleration spectrum, to velocity and then to displacement, the anticipated value of deformation can be obtained. This value is considered to be permanent deformation and is then assessed based on the site-specific implications of damage to the final cover system. Example 6(b) continues the previous pseudo-static analysis into the deformation calculation.

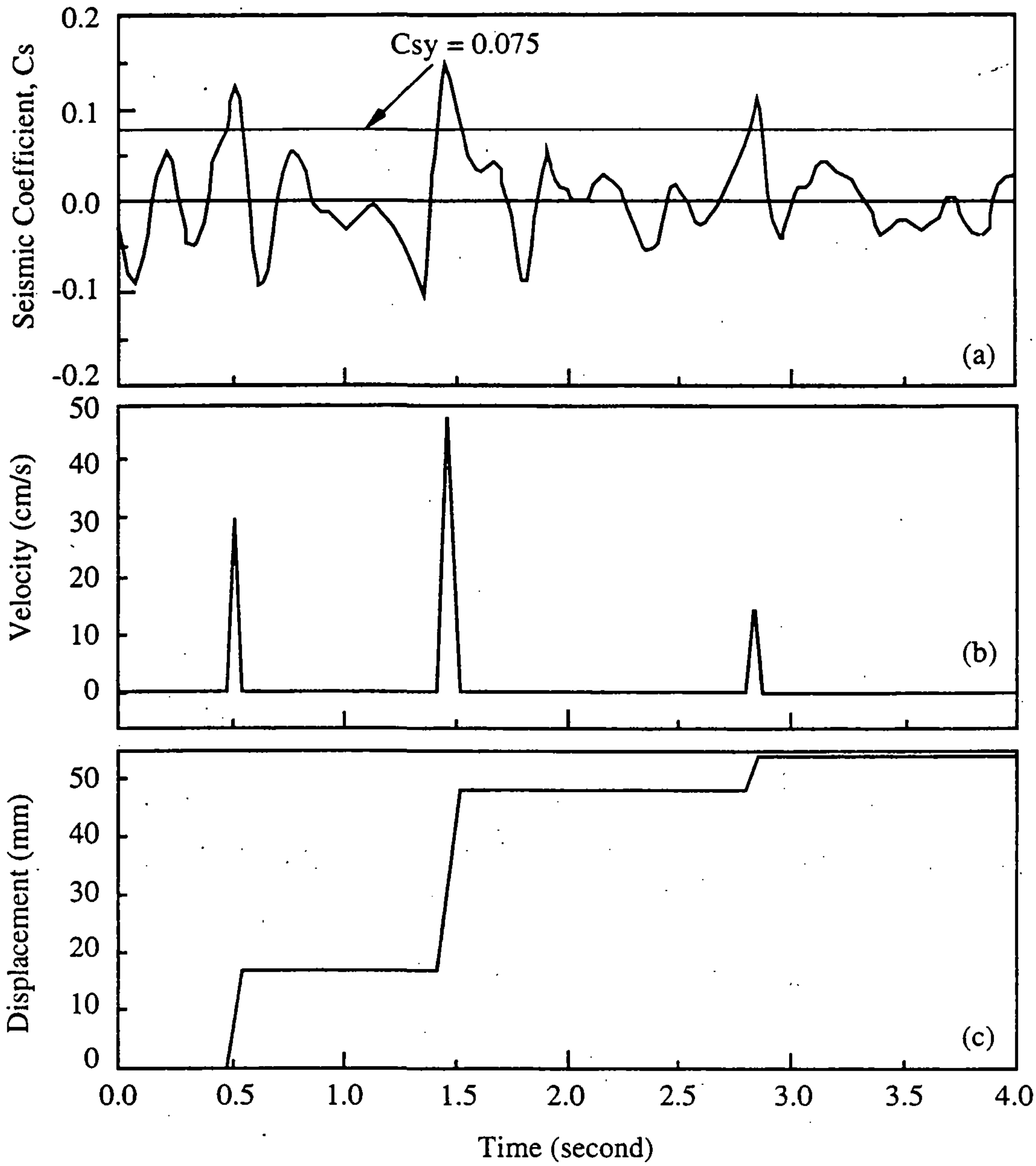


Figure 22 - Design curves to obtain permanent deformation utilizing (a) acceleration, (b) velocity and (c) displacement curves

Example 6(b): Continue the example problem 6(a) and determine the anticipated permanent deformation of the weakest interface in the cover soil system. The site-specific design spectrum is given in Figure 21(a).

Solution: The interface of concern is the cover soil-to-geomembrane for this particular example. With a yield acceleration of 0.075 from Figure 20 and the site-specific design spectrum shown in Figures 21(a), double integration produces Figures 21(b) and (c). The three peaks exceeding the yield acceleration value of 0.075, produce a cumulative deformation of approximately 54 mm. This value is now viewed in light of the deformation capability of the cover soil above the particular geomembrane used at the site.

Comments: An assessment of the implications of deformation (in this example it is 54 mm) is very subjective. For example, this problem could easily have been framed to produce much higher permanent deformations. Such deformations can readily be envisioned in high seismic-prone areas. At the minimum in such an assessment of cover soil systems, the concerns for appurtenances and ancillary piping must be addressed.

6.0 Summary

This report has focused on the mechanics of analyzing slopes as part of final cover systems on engineered landfills, abandoned dumps and remediated waste piles. It also applies to drainage soils placed on geomembrane lined slopes beneath the waste, at least until solid waste is placed against the slope. Design curves in all of the sections have resulted in global FS-values. Each section was presented from a designer's perspective in transitioning from the simplest to the most advanced. Table 1 summarizes the factors-of-safety of the similarly framed example problems so that insight can be gained from each of the conditions analyzed. Throughout the chapter, the inherent danger of building a relatively steep slope on a potentially weak interface material, oriented in the exact direction of a potential slide, should have been apparent.

Table 1 - Summary of Slope Stability Analyses for Similar Conditions per Example Problems in this Section

Example	Condition	FS-value
1	standard example*	1.25
2(a)	equipment up-slope	1.24
2(b)	equipment down-slope	1.03
3	tapered thickness	1.57
4	veneer reinforcement	1.57
5	seepage	0.93
6	seismic	0.94

*30 m long slope, sand cover soil of 300 mm thickness, $\phi = 30$ deg., on a geomembrane with $\delta = 22$ deg., at a slope angle $\beta = 18.4$ deg.

The standard example was purposely made to have a low, but not at failure, factor-of-safety. The FS-value was seen to decrease with construction equipment on the slope, particularly when moving down-slope. This situation should be avoided. While the techniques of a tapered cover soil thickness and veneer reinforcement using geosynthetics both result in significant improvement of the slope's stability, both have implications. The former being additional soil cost and toe space requirements, the latter being the cost of the geosynthetics.

Other options available to the designer to increase the stability of a given situation are as follows:

1. Decrease the slope angle either uniformly by a constant amount, or gradually from the toe to the crest, i.e., crown the cover soil topography into a domed shape.
2. Interrupt long slope lengths with intermediate benches, or even berms if erosion conditions so warrant.
3. Provide for discrete zones of the cover surface topography, each with its separate grading pattern. Thus one will have an accordion folded type of grading scheme, instead of a uniform final topography.
4. Provide for a temporary cover until the majority of the subsidence occurs in the waste mass (perhaps waiting for 5 to 10 years), which has the effect of decreasing the toe-to-crest elevation difference. Construct the final cover at that time.

5. Work with lower than typical factors-of-safety during the post-closure period. If a slide occurs, design the cross section so that the barrier layer is not effected and then repair the slope accordingly.

Lastly, the decrease in FS-value from seepage forces and from seismicity were both illustrated. Clearly, seepage forces should be avoided by providing proper drainage above the barrier layer. On the other hand, seismicity is site-specific and must be dealt with accordingly. In Appendix "A", variations of the above dealing with multiple lined slopes are addressed. It is seen that these problems form a subset of the veneer reinforcement design model presented in Section 3.4.

We conclude with a discussion on factor-of-safety (FS) values for cover soil situations. Note that we are referring to the global FS-value, not partial FS-values which necessarily must be placed on geosynthetic reinforcement materials when they are present. In general, one can consider global FS-values to vary in accordance with the site specific issue of required service time (i.e., the duration) and the implication of a slope failure (i.e., the concern). Table 2 gives the general concept in qualitative terms.

Table 2 - Qualitative Rankings for Global Factor-of-Safety Values in Performing Stability Analyses of Final Cover Systems

Duration → ↓Concern	Temporary	Permanent
Noncritical	Low	Moderate
Critical	Moderate	High

Using the above as a conceptual guide, the authors recommend the use of the minimum global factor-of-safety values listed in Table 3, as a function of the type of underlying waste.

Table 3 - Recommended Global Factor-of-Safety Values in Performing Stability Analyses of Final Cover Systems

Type of Waste→ ↓ Ranking	Hazardous Waste	Nonhazardous Waste	Abandoned Dumps	Remediated Waste Piles
Low	1.4	1.3	1.4	1.2
Moderate	1.5	1.4	1.5	1.3
High	1.6	1.5	1.6	1.4

It is hoped that the above values give reasonable guidance in final cover slope stability decisions, but it should be emphasized that regulatory agreement and engineering judgment is needed in many, if not all, situations.

7.0 References

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Appendix "A" - Slopes with Multiple Interfaces

The various examples presented in the main body of this report are felt by the authors to cover the requisite technical knowledge to safely design linear slopes containing geosynthetic inclusions. The general situation encountered, and developed accordingly, was a cover soil on the surface of a geomembrane, where the interface friction angle between the two materials was low enough to put the cover soil in jeopardy of sliding. A number of variations on this theme (equipment loading, tapered cover soils, veneer reinforcement, seepage effects and seismic effects) were analyzed and illustrated by example problems. Yet, there are literally hundreds of different cross-sections of geosynthetically lined slopes that one can envision. A particular situation which is common (and has not yet been considered) is where the low interface shear strength layer is located <u>beneath</u> the uppermost soil-to-geosynthetic layer. The following situations come to mind and apply equally to liner systems beneath solid waste and cover systems above solid waste:

- Geotextile cushion (for protection purposes) placed above a smooth geomembrane.
- Geomembrane (smooth or textured) placed above a clay soil (CCL or GCL) as with a composite liner.
- Geotextile/geomembrane (usually textured) placed above a clay soil (CCL or GCL), thus a combination of the previous two situations.
- Geotextile/geomembrane (textured)/geonet composite/geomembrane (textured) placed above a clay soil (CCL or GCL) as in a double liner situation.

These situations are illustrated in Figures A-1 to A-4, respectively. In each case, there is the likelihood for a low shear strength interface to be located <u>beneath</u> the uppermost geosynthetic surface. When such an interface has a friction angle lower than the slope angle (i.e., when $\delta < \beta$), tension will be induced in the overlying geosynthetic(s). This tension will cause deformation in the overlying geosynthetic(s) or anchorage pullout unless the overlying geosynthetic(s) are restrained. In such cases of adequate restraint, the deformations can cause tensile stresses that are sufficiently large so as to cause a wide width tensile failure and subsequent sliding of all of the overlying materials.

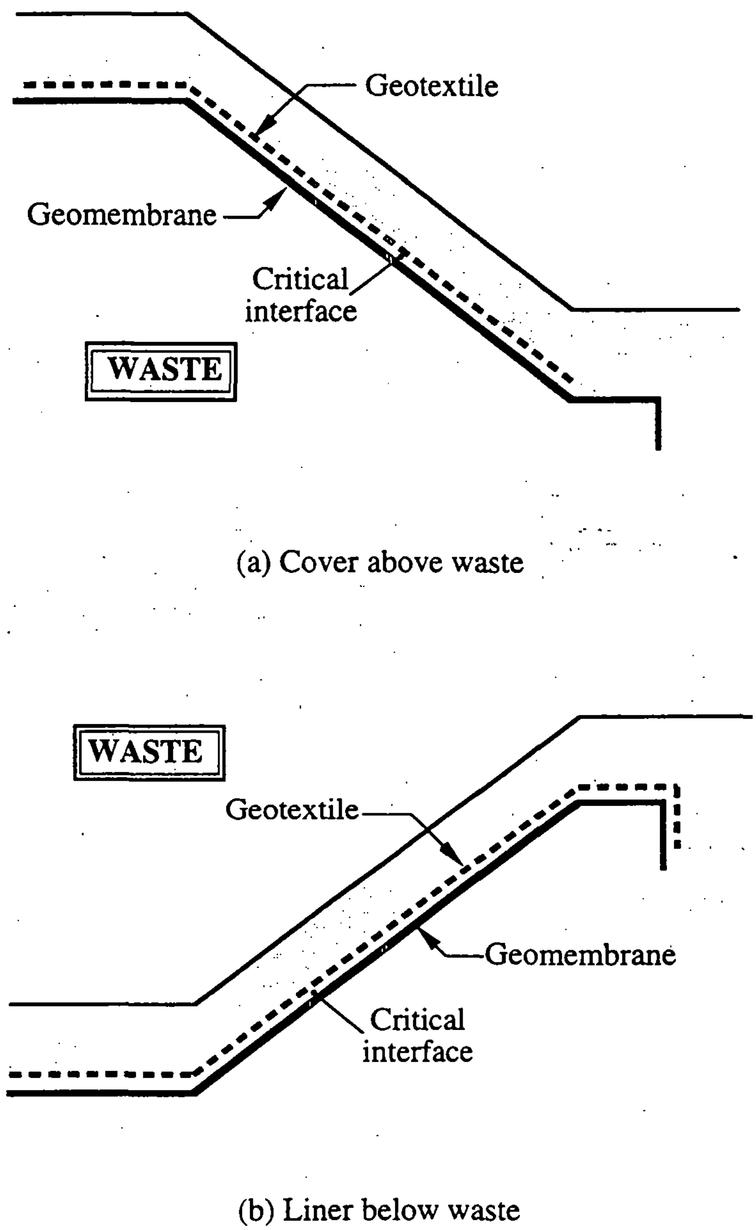
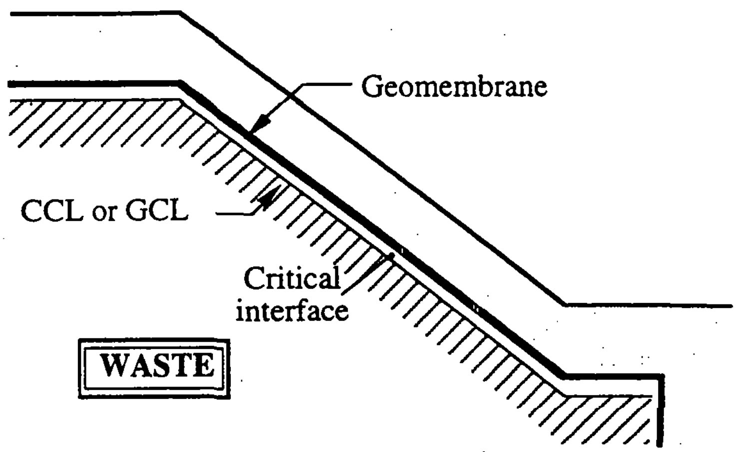
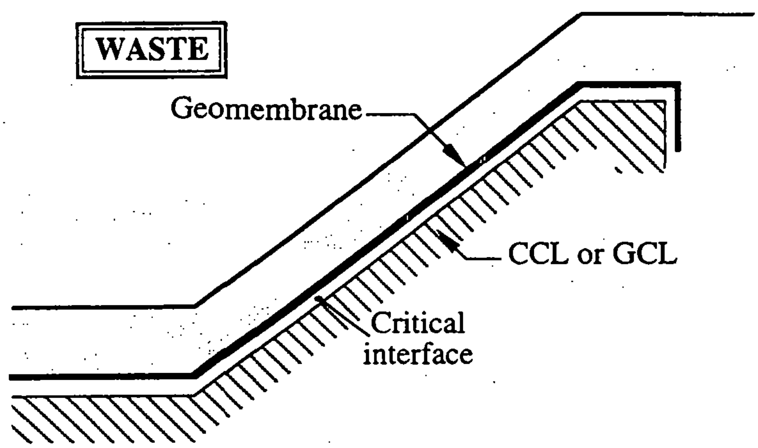


Figure A-1 - Geotextile cushion above a geomembrane

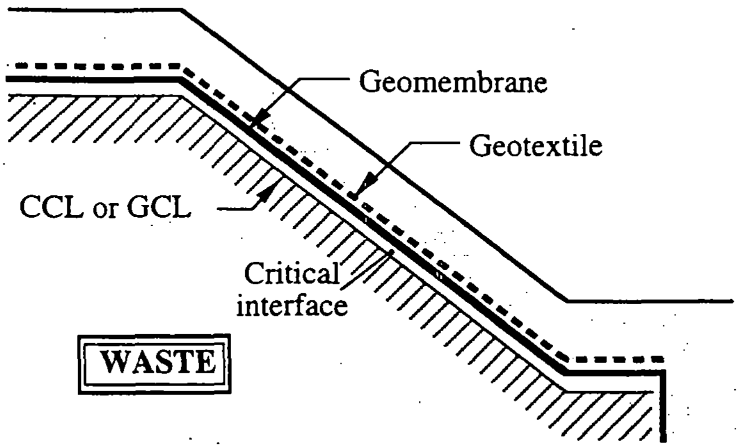


(a) Cover above waste

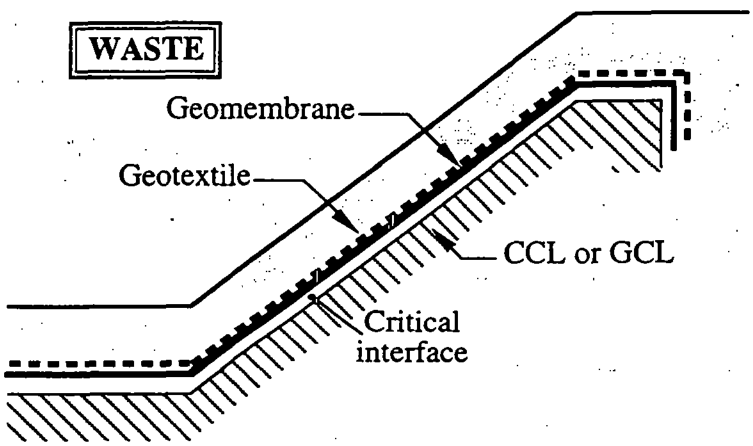


(b) Liner below waste

Figure A-2 - Composite liner consisting of a geomembrane above a compacted clay liner (CCL) or geosynthetic clay liner (GCL)

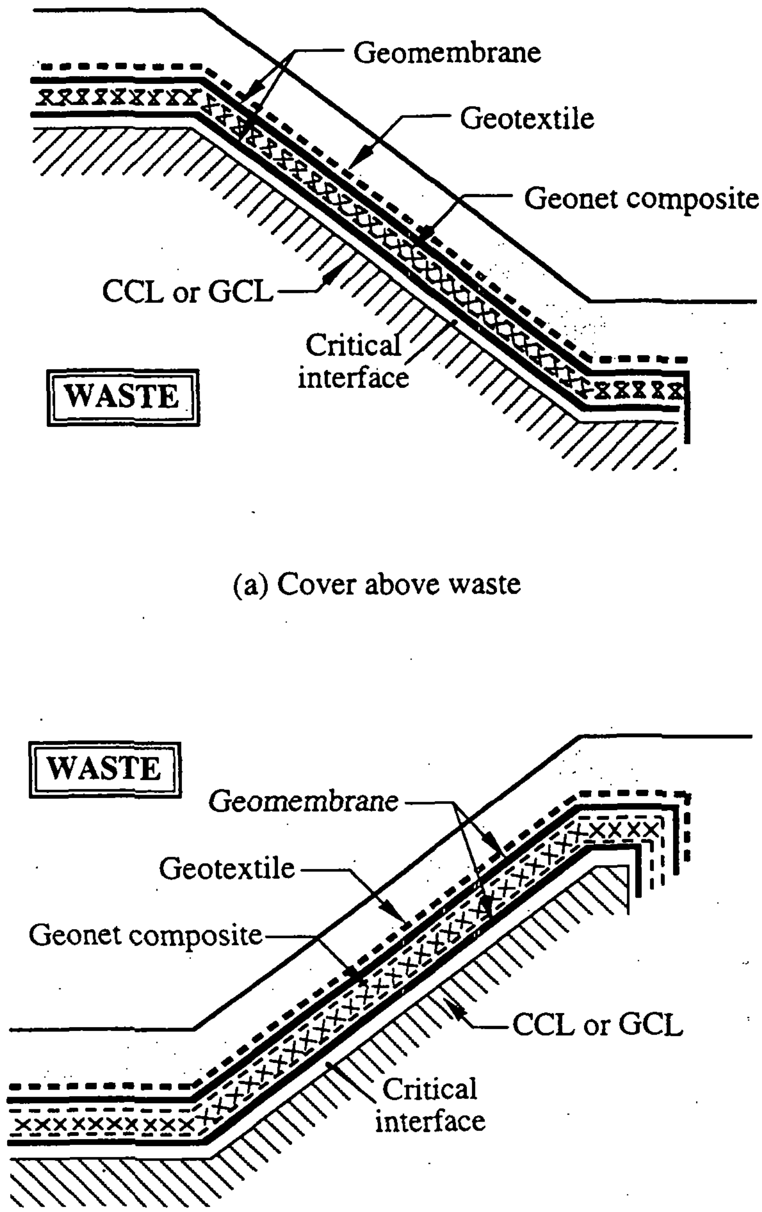


(a) Cover above waste



(b) Liner below waste

Figure A-3 - Geotextile/geomembrane above a compacted clay liner (CCL) or geosynthetic clay liner (GCL)



(a) Cover above waste

(b) Liner below waste

Figure A-4 - Geotextile/geomembrane/geonet composite/geomembrane above a compacted clay liner (CCL) or geosynthetic clay liner (GCL)

Before beginning with numeric problems, however, it should be emphasized that all of the cross sections just presented can be criticized if any of the interface angles fall below the soil slope angle. Even though the geosynthetic(s) above the low interface angle material can possibly maintain the slope from actually failing, some amount of tensile stress will be induced in the overlying geosynthetic(s). *This is contrary to the desire of most design engineers (and manufacturers) and poses regulatory concerns as well.* In essence what is being done in all such cases where $\delta < \beta$ is that the upper geosynthetic(s) is acting as de-facto veneer reinforcement.

When the veneer reinforcement example was given in section 3.4 (recall problem #4), the reinforcement was provided *intentionally* with geogrids or high strength geotextiles in the cover soil. With the cases illustrated in Figures A-1 to A-4, the geosynthetic(s) acts as *non-intentional* reinforcement. Clearly these are situations which should be avoided and, if not possible to avoid, should be challenged insofar as the appropriate design is concerned. Furthermore, an overt acceptance of the implications of tensile stresses being induced into the effected geosynthetics must be assumed.

There are some important distinctions and complications between the analyses to be provided in this Appendix and the analysis given in section 3.4 for veneer reinforcement. They are the following:

- The interface shear strength will be geosynthetic-to-geosynthetic or geosynthetic-to-underlying clay soil, not overlying drainage soil-to-geosynthetic as illustrated for geogrid reinforcement. The reason is that soil strike-through cannot occur for a geotextile or geomembrane, while it can for a geogrid.
- It will be assumed that there is at least one interface beneath the uppermost geosynthetic layer that has a friction angle less than the slope angle, i.e., $\delta < \beta$.
- The assumption that will be made in this appendix is that the overlying geosynthetic(s) is fixed within its anchor trench, to the degree that the full strength of the geosynthetic(s) can be mobilized without pullout. Alternatively, an equal and opposite force can be provided by virtue of the symmetry of the situation, e.g., in a symmetrically crowned

cover soil above the waste. If the assumption of full restraint is not met, the cover soil and geosynthetics will simply slide down slope on the material with the lowest interface shear strength.

- In all cases of nonintentional reinforcement, the full tensile strength of the overlying geosynthetic(s) is capable of being mobilized. Thus there are no partial factors-of-safety on any of the geosynthetics that are involved, i.e., $\prod (FS_P) = 1.0$. If one would purposely use a high strength composite geotextile or scrim reinforced geomembrane as intentional veneer reinforcement (but now located deeper in the cross section than illustrated in section 3.4) one must apply partial factors-of-safety, e.g., $\prod (FS_P) = 2.0, 3.0$ or 4.0. Note that in the example in section 3.4 using intentionally placed geogrid reinforcement, the value of $\prod (FS_P)$ was equal to 4.0.
- When multiple geosynthetics are involved above the low interface shear strength material, strain compatibility of the different materials must be considered. This means that the first geosynthetic to fail in tension will be the material with the lowest strain at failure. This, in turn, will limit the strength available by other geosynthetics which can only sustain stress up to the failure strain of the limiting geosynthetic.

So as to parallel the tutorial concept developed in the main body of the report, we will keep the hypothetical situations as constant as possible, e.g.,

- The thickness of drainage soils over liners <u>beneath</u> the waste will be 300 mm at a soil unit weight of 18.0 kN/m^3. This is the same as in the previous examples.
- The thickness of cover soils over liners <u>above</u> the waste will be 1000 mm at a soil unit weight of 18.0 kN/m^3. This is a new class of examples for this report. The reason is that a number of slides of cover soils on closed landfills have occurred in the modes to be considered.
- The slopes of both liner systems and cover systems will be 3(H)-to-1(V), i.e., $\beta = 18.4$ deg.
- The uninterrupted length of both liner system and cover system slopes will be 30 m.

- The lowest interface angle of shearing resistance within the cross section will be arbitrarily set at 10 deg. This value is probably a lower bound friction angle for most interfaces; thus the resulting factors-of-safety that are calculated will generally be lower than found in practice. However, this is obviously a site-specific and material-specific situation.
- The necessity of accurately assessing the interface friction angle by appropriate laboratory testing can not be over-emphasized. (Recall Section 2.2 in the main body of the report).
- The possible adhesion of soil to geosynthetic or geosynthetic-to-geosynthetic will be discounted. This was done in all previous problems and will be continued in this appendix as well. As will be seen, however, the analytic formulation is such that an adhesion value can readily be included in the analysis.

In all of the examples to follow, the analysis is exactly as was developed and shown in section 3.4 for veneer reinforcement. The relevant equation is in the form of $ax^2+bx+c=0$ which in our case using FS-values is:

$$a(FS)^2 + b(FS) + c = 0 \tag{13}$$

where

$$\begin{aligned} a &= (W_A - N_A\cos\beta - T\sin\beta)\cos\beta \\ b &= -[(W_A - N_A\cos\beta - T\sin\beta)\sin\beta\tan\phi + (N_A\tan\delta + C_a)\sin\beta\cos\beta \\ &\qquad + \sin\beta(C + W_p\tan\phi)] \\ c &= (N_A\tan\delta + C_a)\sin^2\beta\tan\phi \end{aligned} \tag{36}$$

The resulting FS-value is then obtained from the following equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \tag{15}$$

The critical parameter in this Appendix (as it was in section 3.4) is the value "T" in equation

(36). Of course, "T" is the allowable wide width tensile strength of the geosynthetic(s) involved above the lowest friction interface material. This value of wide width strength is obtained from ASTM D4595 for geotextiles, geonets and geocomposites and ASTM D4885 for geomembranes, which results in the value of T_{ult}. In turn, T_{ult} is generally reduced by partial factors-of-safety to arrive at the allowable strength. The equation for geotextiles and geogrids is as follows.

$$T_{allow} = T_{ult}\left(\frac{1}{FS_{ID} \times FS_{CR} \times FS_{CBD}}\right) \tag{37}$$

where

T_{allow} = allowable reinforcement strength

T_{ult} = ultimate (as-manufactured) value of reinforcement strength

FS_{ID} = partial factor-of-safety for installation damage

FS_{CR} = partial factor-of-safety for creep

FS_{CBD} = partial factor-of-safety for chemical/biological degradation

The parallel equation for other geosynthetics (geomembranes, geonets, geocomposites and geosynthetic clay liners) has not yet been formulated in the above format. For the purpose of this report the value of T_{ult}, for all geosynthetics, will be reduced by a collective Π (FS_p)-value.

Example A-1 - It is very common to use a geotextile as cushion material for the protection of an underlying geomembrane. This is almost always the case in liner systems beneath the waste and is common above the waste as well. There are many geotextiles that can be used, however, the usual geotextile is a needle punched nonwoven in the mass per unit area range of 400 to 1000 g/m^2. When placed as shown in Figure A-1, the friction angle of the geotextile to the soil above is generally high, e.g., $\delta \geq 25$ deg. However, the friction angle to the underlying geomembrane can be quite low. This is particularly the case for smooth surface geomembranes, where the angle can be as low as 10 deg. This is the situation that will be considered.

Given: For a 30 m long slope at 3(H)-to-1(V), i.e., 18.4 deg., lined with a geotextile over a geomembrane with an interface friction angle of 10 deg., what are the factors-of-safety for the following nonwoven heat bonded (NW-HB), nonwoven needle punched (NW-NP), woven slit film (W-SF), woven monofilament (W-MonoF) and woven multifilament (W-MultiF) geotextiles? Do the problems for 300 mm and 1000 mm cover soil thickness at a soil unit weight of 18 kN/m^3.

No.	Geotextile Type	Weight	Wide Width Strength
GT-1	NW-HB	140 g/m^2	10 kN/m
GT-2	NW-NP	540 g/m^2	25 kN/m
GT-3	W-SF	260 g/m^2	40 kN/m
GT-4	W-MonoF	480 g/m^2	80 kN/m
GT-5	W-MultiF	850 g/m^2	210 kN/m

Solution: Using equations (13), (36) and (15) (with no partial factors-of-safety applied, i.e., Π (FS_P) = 1.0), the calculated global factors-of-safety are as follows:

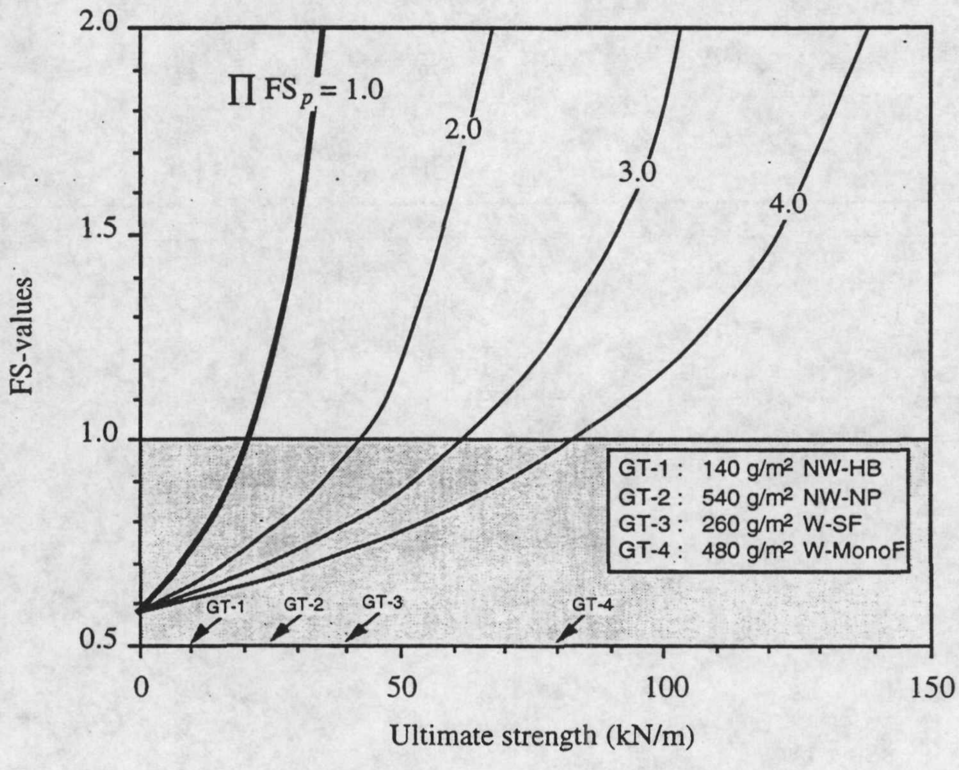
No.	$FS_{300\ mm}$	$FS_{1000\ mm}$
GT-1	0.72	0.74
GT-2	**1.15**	**0.82**
GT-3	2.97	0.93
GT-4	∞	1.42
GT-5	∞	∞

Note that geotextile GT-2 (the needle punched nonwoven) is the typical protection geotextile used in this application and its FS-value is inadequate irrespective of the cover soil thickness.

Also note that with the woven geotextiles, GT-3, GT-4 and GT-5, the FS-values can reach an acceptable value and in such cases one is providing veneer reinforcement in the same way as with the geogrid reinforcement of section 3.4. Differences, however, are that thin woven geotextiles do not provide much cushioning action and that no partial factors-of-safety have been applied (see next problem).

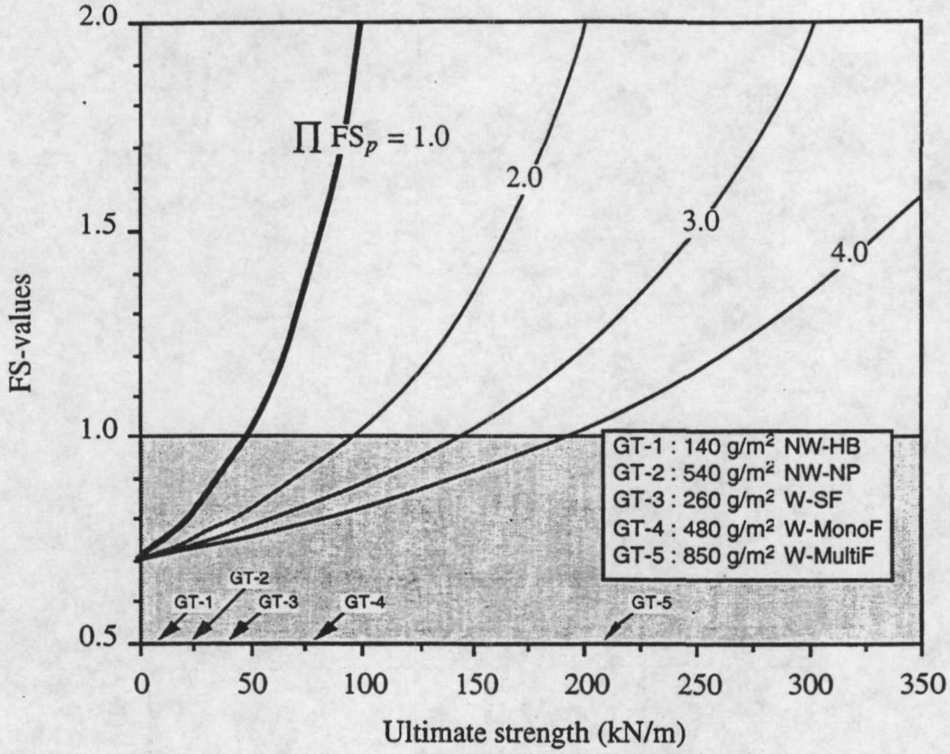
Given: Redo each of the previous problems, but now use partial factors-of-safety of 2.0, 3.0 and 4.0 on the ultimate strength of the geotextiles and plot the various responses. Also show on the graphs the response of the previous problem where no partial factors-of-safety were applied, i.e., with $\prod (Fs_p) = 1.0$.

Solution: See graphs in Figure A-5 which are self explanatory.



(a) Thickness of cover soil = 300 mm

Figure A-5 - Graphic solution of example A-1.



(b) Thickness of cover soil = 1000 mm

Figure A-5 - Graphic solution of example A-1. (*cont'd.*)

Example A-2 - The barrier system of choice in most federal regulations, e.g. in the USA and Germany, is generally considered to be a composite liner. Two possibilities exist; a geomembrane over a compacted clay liner (GM/CCL), or a geomembrane over a geosynthetic clay liner (GM/GCL), see Figure A-2. In both cases, the interface can result in low shear strength. For a GM/CCL, the high water content of the clay, perhaps aggravated by pore water expulsion, can give low interface friction values. For a GM/GCL, the migration of hydrated bentonite through the covering geotextile can give low interface friction values. Thus, both situations require care in material selection (textured geomembranes and needle punched nonwoven geotextiles help considerably) and in design. Following is a composite liner slope stability example.

Given: For a 30 m long slope at 3(H)-to-1(V), i.e., 18.4 deg., lined with a geomembrane over a

CCL or GCL with interface friction angle of 10 deg., what are the factors-of-safety for the following very flexible polyethylene (VFPE), high density polyethylene (HDPE) and scrim reinforced polypropylene (PP-R) geomembranes? Do the problems for 300 mm and 1000 mm cover soil thicknesses at a soil unit weight of 18 kN/m^3.

No.	Geomembrane Type	Thickness	Wide Width Strength
GM-1	VFPE	1.0 mm	15 kN/m
GM-2	HDPE	1.5 mm	25 kN/m
GM-3	PP-R	1.1 mm	40 kN/m

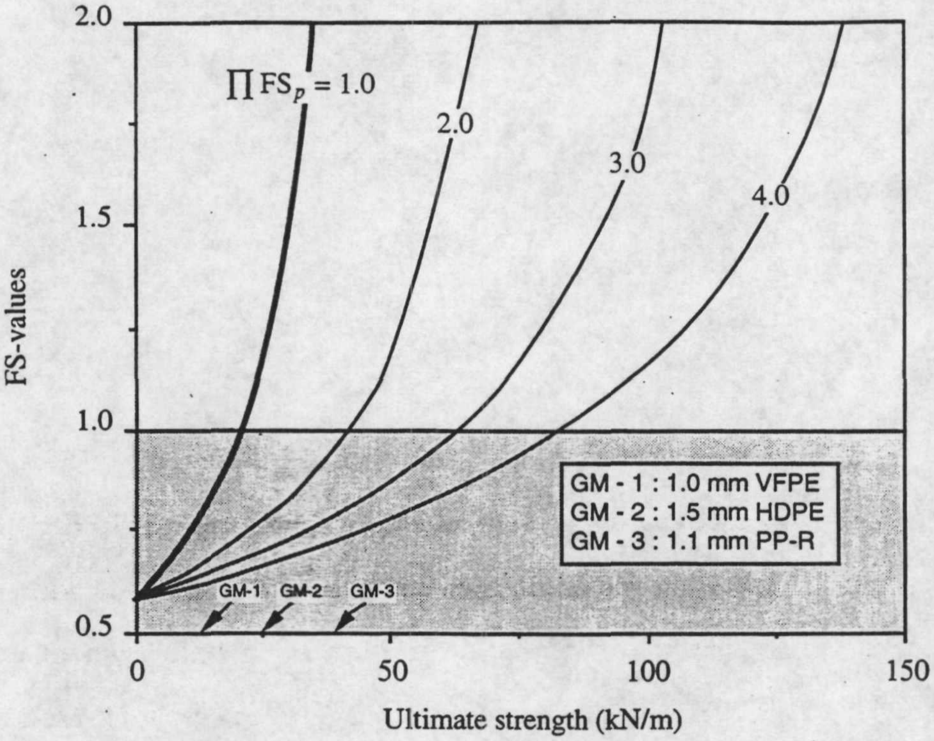
Solution: Using equations (13), (36) and (15) (with no partial factors-of-safety applied, i.e., $\prod(FS_p) = 1.0$), the calculated global factors-of-safety are as follows:

No.	$FS_{300\ mm}$	$FS_{1000\ mm}$
GM-1	0.82	0.77
GM-2	1.15	0.82
GM-3	2.97	0.93

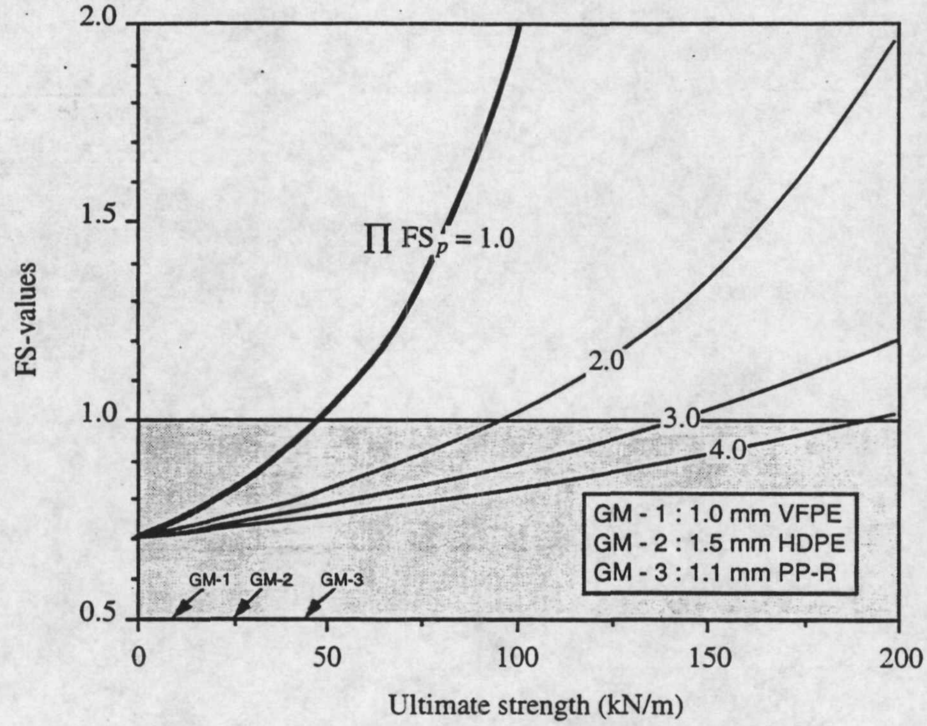
As readily seen, the generally unacceptable FS-values that arise when using geomembranes as reinforcement materials are obvious. The possible exception is PP-R which is scrim reinforced. Indeed, as the fabric reinforcement strength within the geomembrane increases one can have stability, but the concept of a composite barrier/reinforcement material is somewhat questionable. Such a design must be carefully considered. If such a reinforced geomembrane is desired, however, partial factors-of-safety should be included as illustrated in the following problem.

Given: Redo each of the previous problems, but now use partial factors-of-safety of 2.0, 3.0 and 4.0 on the ultimate strength of the geomembranes and plot the various responses. Also show on the graphs the response of the previous problem where no partial factors-of-safety were applied, i.e., $\prod (FS_p) = 1.0$.

Solution: See graphs in Figure A-6 which are self explanatory.



(a) Thickness of cover soil = 300 mm



(b) Thickness of cover soil = 1000 mm

Figure A-6 - Graphic solution of example A-2.

Example A-3 - The combination of the previous two examples gives rise to a situation where a geotextile cushion placed over a geomembrane, is underlain by a CCL or GCL, see Figure A-3. If the GM/CCL or GM/GCL interface friction angle is low, both of the overlying geosynthetics (geotextile and geomembrane) will be placed in tension. Yet, they are acting as individual units since they are not bonded together. Thus, their wide-width stress vs. strain behavior must be laboratory evaluated and carefully considered. Examples using different geotextiles (as previously defined) against different geomembranes (as previously defined) follows.

Given: For a 30 m long slope at 3(H)-to-1(V), i.e., 18.4 deg., lined with a geotextile over a geomembrane which is on a CCL or GCL, with interface friction angle of 10 deg., what are the factors-of-safety for the following geotextile/geomembrane variations? The wide-width stress vs. strain response curves of the different geotextiles along with each of the different geomembranes (VFPE, HDPE and PP-R) are provided in Figures (A-7), (A-8) and (A-9), respectively. Do the problems for 300 mm and 1000 mm cover soil thicknesses at a soil unit weight of 18 kN/m^3.

No.	GM Type	Geomembrane ultimate strength	Geotextile* at ultimate strength
GM-1	1.0 mm VFPE	15 kNm	GT-1 @ 10 kN/m GT-2 @ 25 kN/m GT-3 @ 40 kN/m GT-4 @ 80 kN/m
GM-2	1.5 mm HDPE	25 kN/m	GT-1 @ 10 kN/m GT-2 @ 25 kN/m GT-3 @ 40 kN/m GT-4 @ 80 kN/m
GM-3	1.1 mm PP-R	40 kN/m	GT-1 @ 10 kN/m GT-2 @ 25 kN/m GT-3 @ 40 kN/m GT-4 @ 80 kN/m

*GT-1: 140 g/m^2 NW-HB
GT-2: 540 g/m^2 NW-NP
GT-3: 260 g/m^2 W-SF
GT-4: 480 g/m^2 W-MonoF

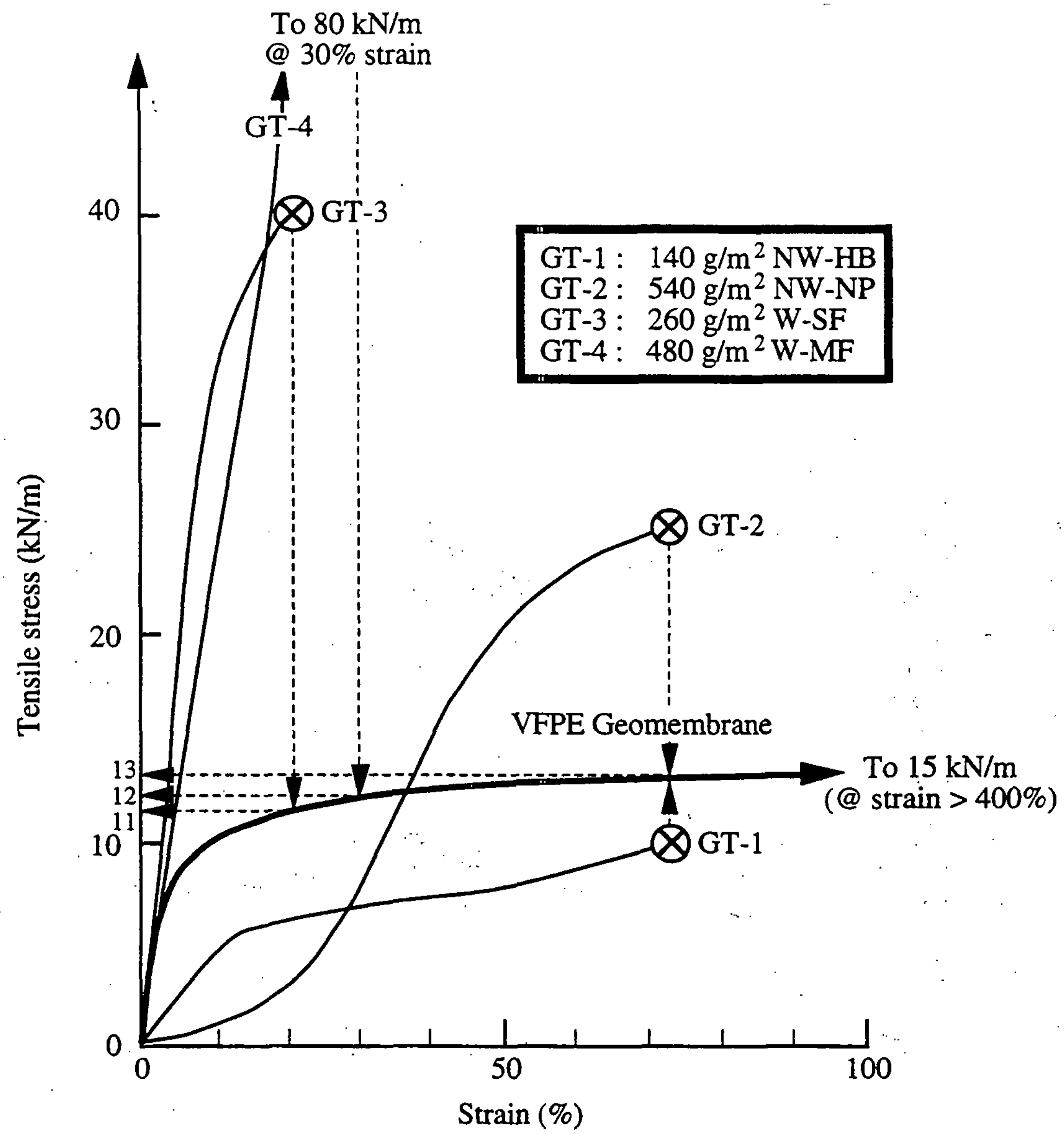


Figure A-7 - The wide-width stress vs. strain behavior of a 1.0 mm VFPE geomembrane and four different geotextiles described in example A-3.

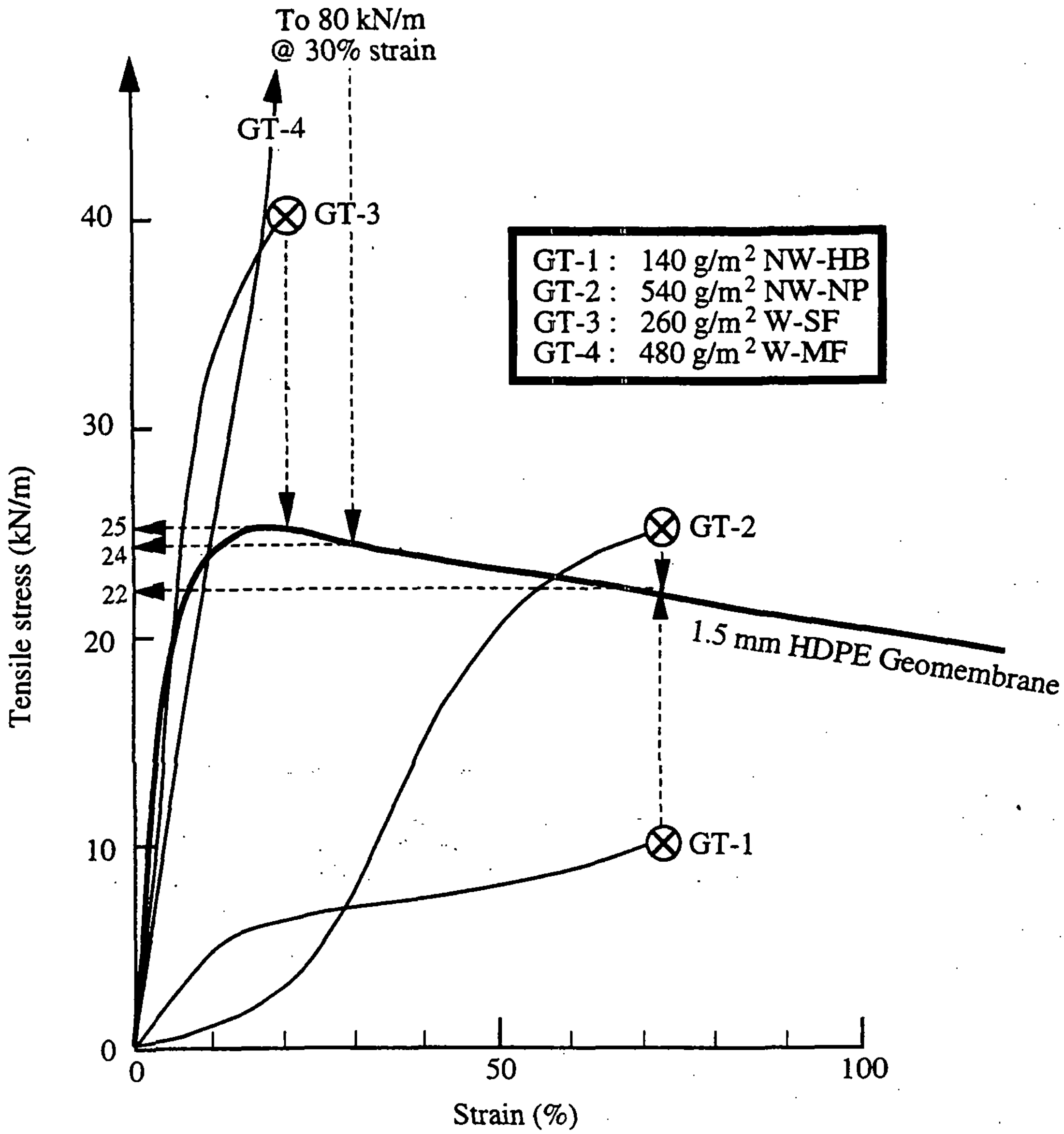


Figure A-8 - The wide-width stress vs. strain behavior of a 1.5 mm HDPE geomembrane and four different geotextiles described in example A-3.

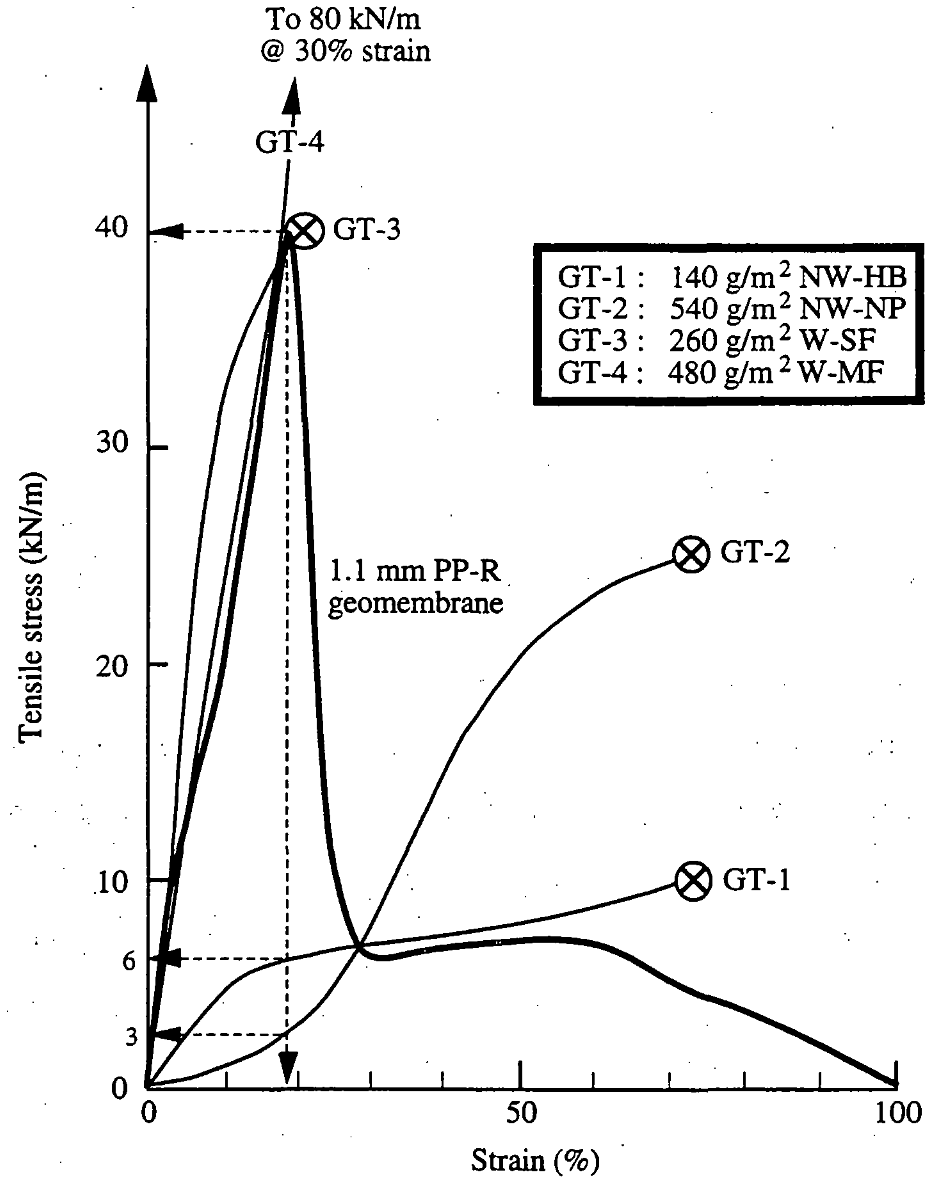


Figure A-9 - The wide-width stress vs. strain behavior of a 1.1 mm PP-R geomembrane and four different geotextiles described in example A-3.

Solution: Using equations (13), (36) and (15), (with no partial-factors-of-safety applied), the calculated global FS-values are given in Table A-1 under the column headings of $\prod (FS_p) = 1.0$. Here it can be seen that global FS-values can indeed reach high values for the 300 mm-thick cover soil. Thus they may be considered for liner systems beneath waste. For the 1000 mm cover soil thickness typical of cover system above the waste, however, the global FS-values are invariably too low. For both situations, however, partial factors-of-safety have not been included. This is clearly not a proper design for cover systems which must have stability throughout their design life.

Given: Redo the previous problems but now use partial factors-of-safety of 2.0, 3.0 and 4.0 on the ultimate strength of the geosynthetics involved. Show the results in tabular form and compare the results to the previous problem where no partial factors-of-safety were applied, i.e., $\Pi\,(FS_p) = 1.0$.

Solution: See the continuation of Table A-1 where the calculated global FS-values steadily decrease as $\Pi\,(FS_p)$ increases. In all cases they eventually reach unacceptable values.

Numerous important behavioral trends can be observed:

- The thickness of the cover soil is a major issue in the calculated FS-values; clearly, thick cover soils are troublesome by their shear mass and are prone to sliding with respect to thinner cover soils.
- For thick cover soils with a low shear strength interface in the barrier system (typically in the cover system), veneer reinforcement within the cover soil will generally be necessary.
- The FS-value increases with increased geotextile strength.
- The geotextile strength can dominate over the strength of the geomembrane.
- There is, however, an increase in FS-value with increased geomembrane strength.

Table A-1 - Calculated global FS-values of example problem A-3.

Geomembrane/ ultimate strength	Geotextile*/ ultimate strength	Composite strength (GM+GT)		Thickness of cover soil = **300 mm**				Thickness of cover soil = **1000 mm**		
			$\Pi(FS_p)$=1.0	$\Pi(FS_p)$=2.0	$\Pi(FS_p)$=3.0	$\Pi(FS_p)$=4.0	$\Pi(FS_p)$=1.0	$\Pi(FS_p)$=2.0	$\Pi(FS_p)$=3.0	$\Pi(FS_p)$=4.0
1.0 mm VFPE/ 15 kN/m	GT-1 / 10 kN/m	13+10=23 kN/m	**1.07**	0.75	0.68	0.65	0.81	0.75	0.73	0.72
	GT-2 / 25 kN/m	13+25=38 kN/m	**2.45**	0.93	0.77	0.71	0.91	0.79	0.76	0.74
	GT-3 / 40 kN/m	11+40=51 kN/m	**> 10**	**1.18**	0.87	0.77	**1.02**	0.83	0.78	0.76
	GT-4 / 80 kN/m	12+80=92 kN/m	**> 10**	**8.13**	**1.50**	**1.07**	**1.70**	0.98	0.86	0.81
1.5 mm HDPE/ 25 kN/m	GT-1 / 10 kN/m	22+10=32 kN/m	**1.61**	0.85	0.73	0.69	0.87	0.77	0.75	0.73
	GT-2 / 25 kN/m	22+25=47 kN/m	**> 10**	**1.09**	0.84	0.75	0.99	0.81	0.77	0.75
	GT-3 / 40 kN/m	25+40=65 kN/m	**> 10**	**1.66**	**1.02**	0.85	**1.18**	0.87	0.80	0.77
	GT-4 / 80 kN/m	24+80=104 kN/m	**> 10**	**> 10**	**1.90**	**1.20**	**2.11**	**1.03**	0.89	0.83
1.1 mm PP-R/ 40 kN/m	GT-1 / 10 kN/m	40+ 6=46 kN/m	**8.13**	**1.07**	0.83	0.75	0.98	0.81	0.77	0.75
	GT-2 / 25 kN/m	40+ 3=43 kN/m	**4.34**	**1.01**	0.81	0.73	0.95	0.80	0.76	0.75
	GT-3 / 40 kN/m	40+40=80 kN/m	**> 10**	**2.97**	**1.24**	0.96	**1.42**	0.93	0.83	0.79
	GT-4 / 80 kN/m	40+40=80 kN/m	**> 10**	**2.97**	**1.24**	0.96	**1.42**	0.93	0.83	0.79

* GT-1 : 140 g/m^2 NW-HB
GT-2 : 540 g/m^2 NW-NP
GT-3 : 260 g/m^2 W-SF
GT-4 : 480 g/m^2 W-MF

Example A-4 - While iterations on the above theme can be easily envisioned, the following example illustrates a relatively common situation, i.e., a double liner system per Figure A-4. In this situation, all of the geosynthetics above the secondary geomembrane-to-CCL or GCL interface will be assumed to have interface friction values higher than 10 deg. The GM/CCL or GM/GCL, interface is assumed to be 10 deg. As with the previous problem, strain compatibility must be considered. The situation is complicated because of the large number of geosynthetics that are involved.

Given: For a 30 m long slope at 3(H)-to-1(V), i.e., 18.4 deg., lined with a double liner system consisting of GT/GM/GC/GM/CCL or GCL, the lowest friction angle is the GM/CCL or GM/GCL, which is 10 deg. What are the global FS-values for the following combination of various geosynthetics? Thus, each of the three different geomembranes [VFPE (textured), HDPE (textured) and PP-R] are located beneath a 540 g/m^2 nonwoven needle punched cushion geotextile and are sandwiching a geocomposite consisting of a geonet which has thermally bonded geotextiles on both sides. The wide-width stress vs. strain response curves of the geotextile and geonet composite, along with each of the three different geomembranes (VFPE, HDPE and PP-R), are provided in Figures (A-10), (A-11) and (A-12), respectively. Do the problems for 300 mm and 1000 mm cover soil thicknesses at a soil unit weight of 18 kN/m^3 with partial factors-of-safety of 1.0, 2.0, 3.0 and 4.0.

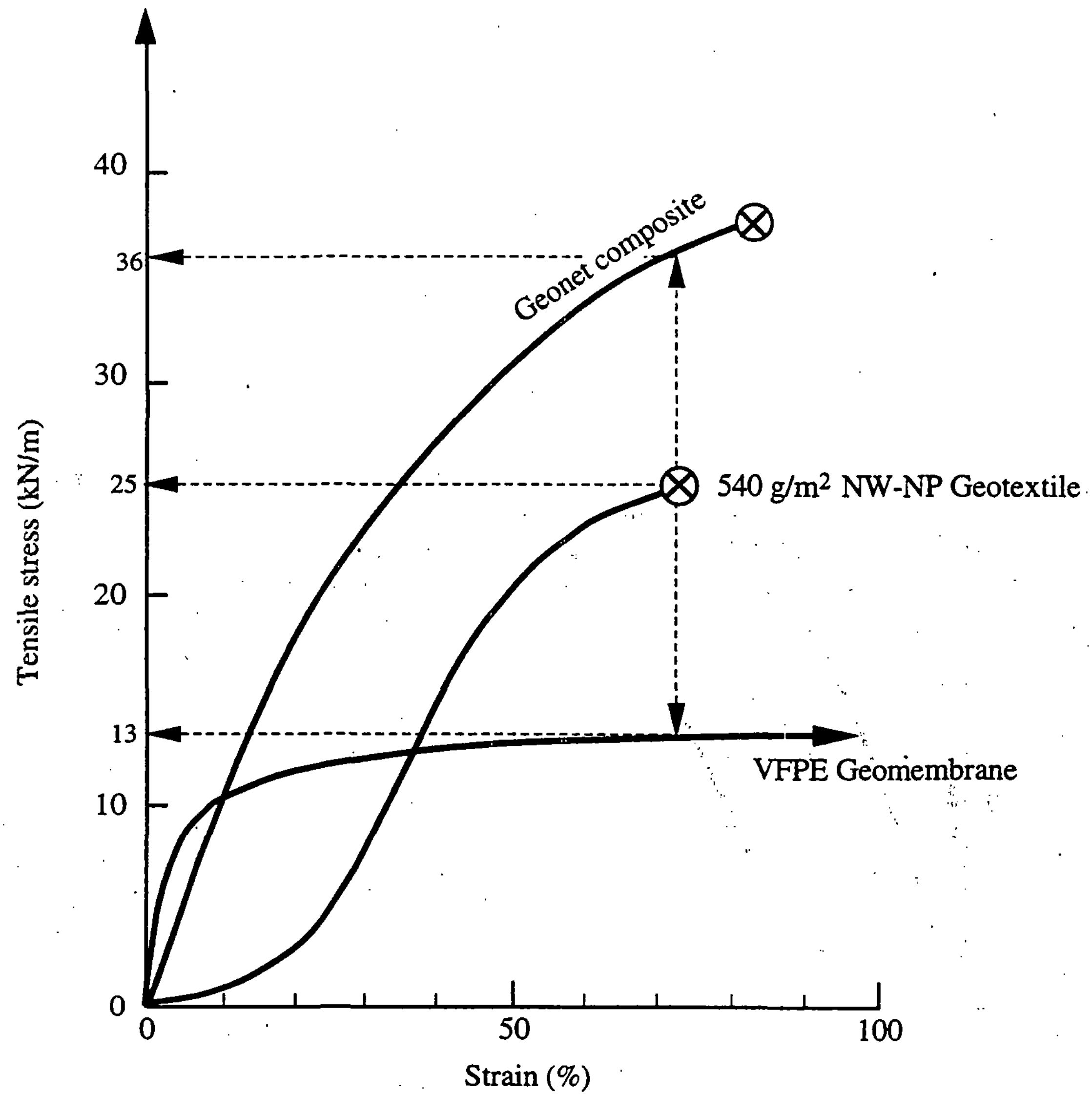


Figure A-10 - The wide-width stress vs. strain behavior of the geotextile, geonet composite and VFPE geomembrane used in example A-4.

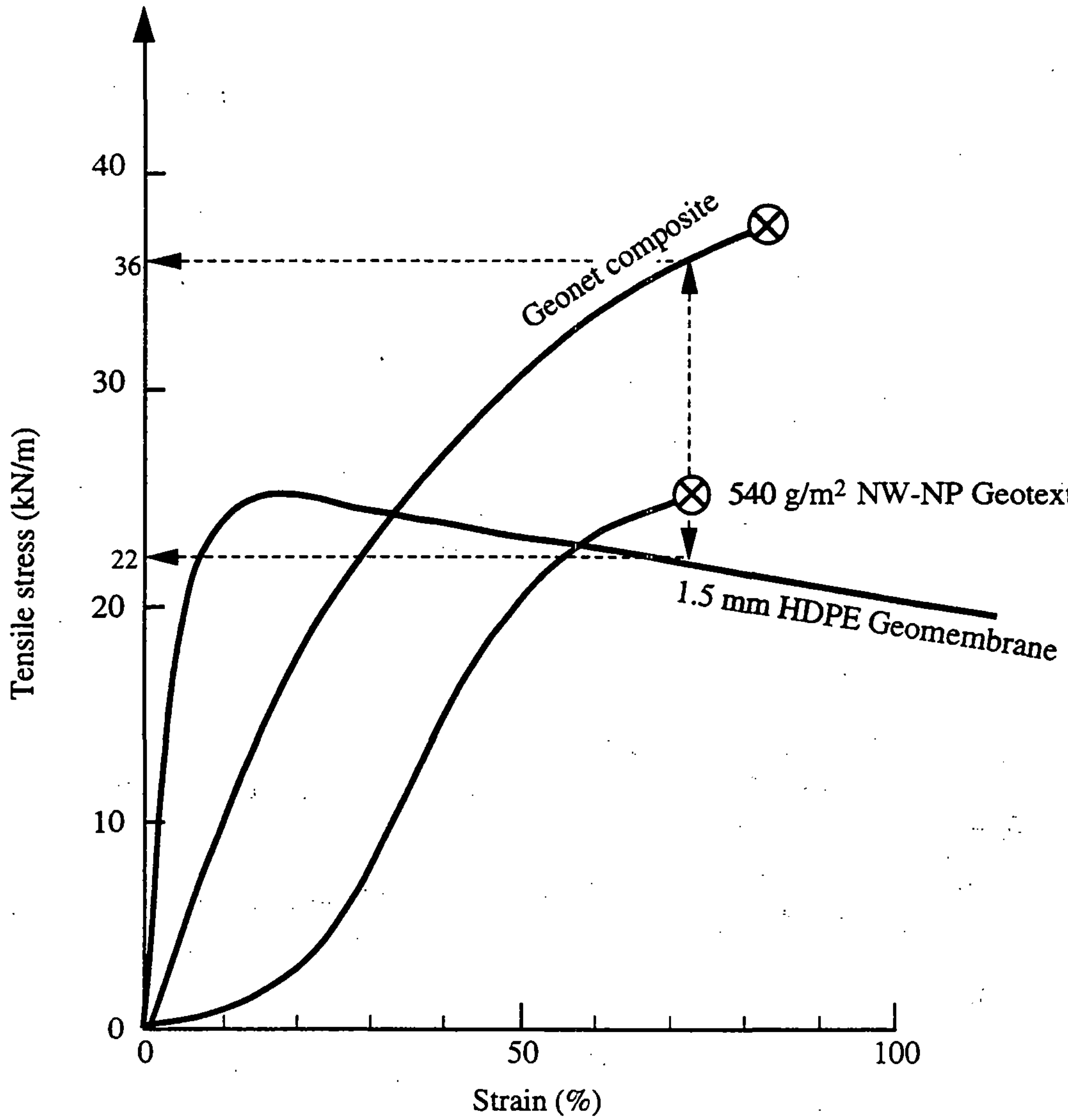


Figure A-11 - The wide-width stress vs. strain behavior of the geotextile, geonet composite and HDPE geomembrane used in example A-4.

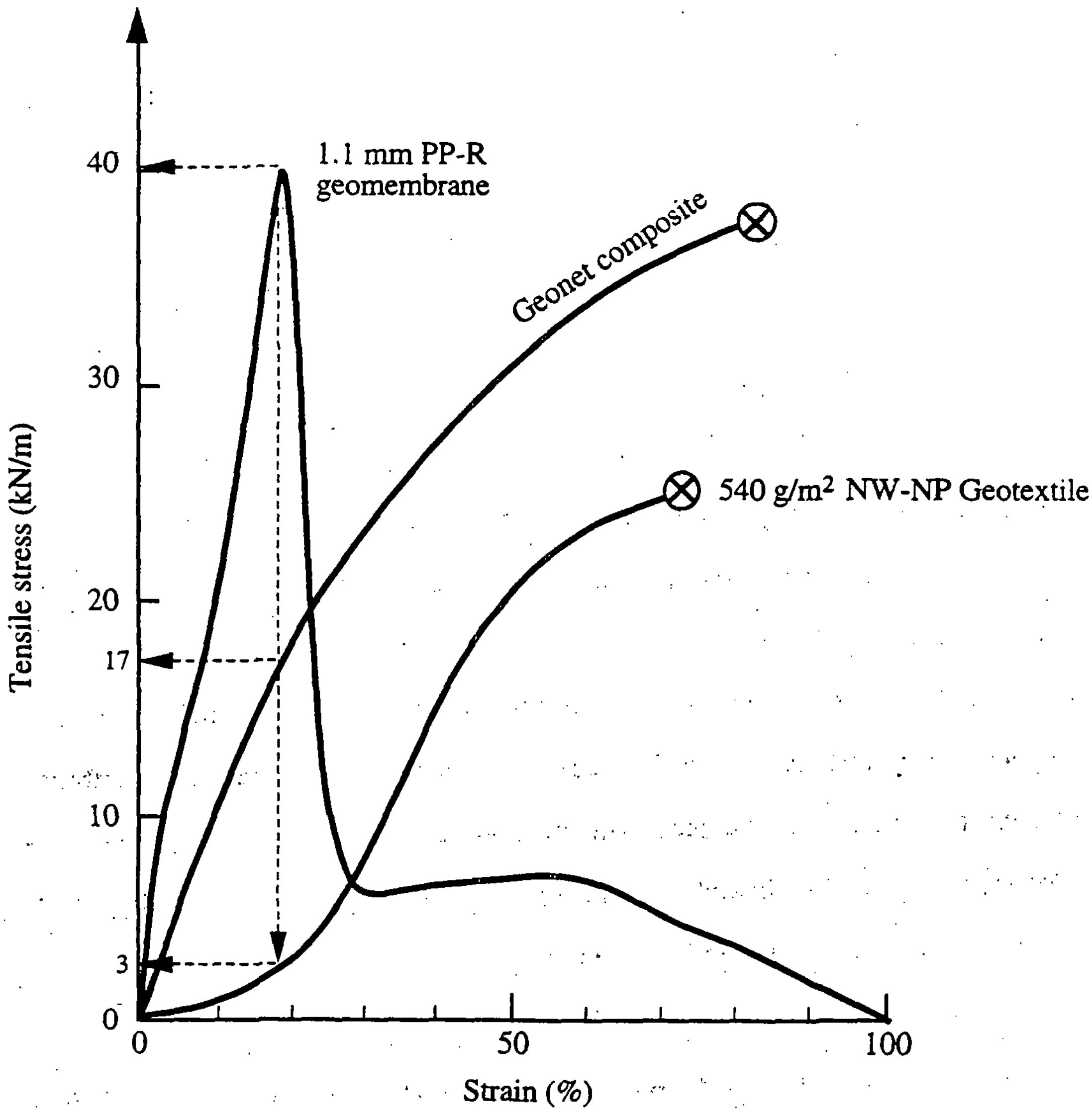


Figure A-12 - The wide-width stress vs. strain behavior of the geotextile, geonet composite and PP-R geomembrane used in example A-4.

Errata for page 78

Solution: Using equations (13), (36) and (15), the calculated global FS-values are as follows:

Thickness of cover soil	Geomembrane Type	Partial factors-of-safety			
		Π(FS*p*)=1.0	Π(FS*p*)=2.0	Π(FS*p*)=3.0	Π(FS*p*)=4.0
300 mm	VFPE	∞	4.71	1.38	1.02
	HDPE	∞	∞	1.94	1.21
	PP-R	∞	∞	1.74	1.15
1000 mm	VFPE	1.57	0.96	0.85	0.80
	HDPE	2.16	1.04	0.89	0.83
	PP-R	1.95	1.01	0.88	0.82

As seen in the above table, the FS-values are generally acceptable when no partial factors-of-safety are applied to the overlying geosynthetics. However, they gradually fall off to unacceptable values when gradually higher partial factors-of-safety are included. Moreover, contrary to what was observed in example A-2, the scrim reinforced polypropylene geomembrane provides little reinforcement in this case. This is due to the strain compatibility consideration. As seen in Figure A-12, the geotextile cushion contributes relatively little before the scrim reinforcement of the PP-R geomembrane breaks.

If one is tempted to accept the reinforcement hypothesis of the respective overlying geosynthetics, the situation must be challenged on the basis of two other considerations:

- Relatively large deformations, to the extent of the mobilized strain in the geosynthetics will be experienced. The implication of these deformations, particularly at the toe of the slopes, must be assessed.
- With such large forces mobilized in the geosynthetics, the resistance within the anchor trench cannot be overlooked. Anchor trench design becomes a necessary complement to the system design. See Koerner and Wayne (1990) and Hulling (1996) for some guidance in this regard.

Appendix "B" - Computer Worksheets

Seven computer worksheets for calculating the FS-values of different cover soil stability problems were constructed using Microsoft Excel (version 5.0) computer software. They are listed as follows.

- Worksheet #1; for uniform thickness cover soil
- Worksheet #2(a); for uniform thickness cover soil with the incorporation of equipment working up slope
- Worksheet #2(b); for uniform thickness cover soil with the incorporation of equipment working down slope
- Worksheet #3; for tapered thickness cover soil
- Worksheet #4; for uniform thickness cover soil with veneer reinforcement
- Worksheet #5(a); for uniform thickness cover soil with horizontal seepage buildup
- Worksheet #5(b); for uniform thickness cover soil with parallel-to-slope seepage buildup
- Worksheet #6; for uniform and/or tapered thickness cover soil with consideration of seismic forces

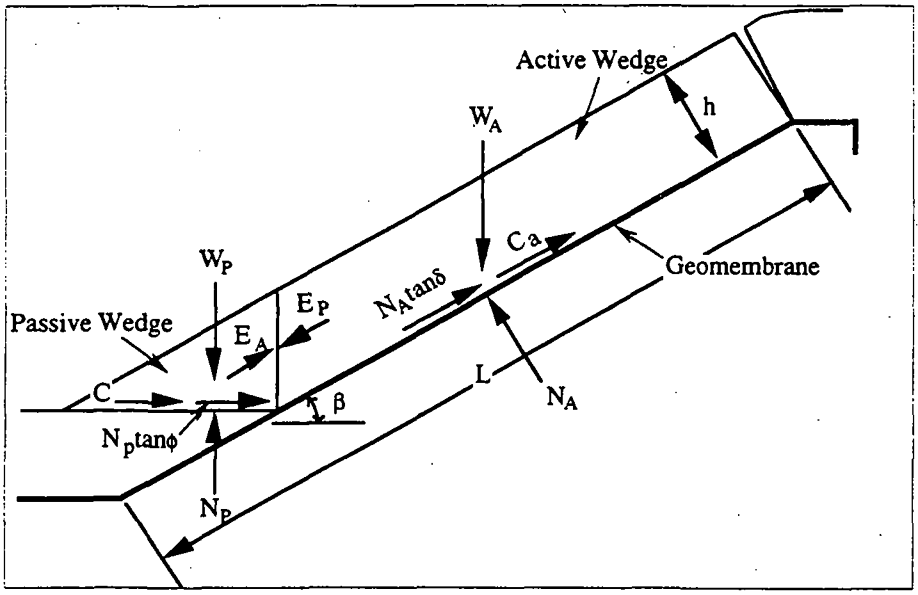
The above worksheets were used extensively during the preparation of this report. All of the design curves presented in the report were generated via these worksheets. The solutions of the example problems presented in this report were also obtained using these worksheets. The computer printouts for the various problems are shown on the following pages.

As seen in the computer printouts, the numbers in boxes are the required input data. The intermediate calculated values, along with the final resulting FS-values, are shown in italic letters. The detailed definitions of the input data can be found in the appropriate sections of the report.

Note that these computer worksheets were designed as tools for a generalized parametric study and/or sensitivity analysis. They should <u>not</u> be used as replacements for detailed, site specific engineering design.

SOIL SLOPE STABILITY ANALYSIS - WORKSHEET #1

Uniform Cover Soil Thickness



Calculation of FS

Active Wedge:

Wa= 156.6 kN

Na= 148.6 kN

Passive Wedge:

Wp= 2.7 kN

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a= 14.8

b= -21

c= 3.5

FS= 1.25

thickness of cover soil = h =	0.30	m				
cover soil slope angle beneathe the geomembrane = β =	18.4	°	*= 0.32*	*(rad.)*		
finished cover soil slope angle = ω =	18.4	°	*= 0.32*	*(rad.)*		
length of slope measured along the geomembrane = L =	30.0	m				
unit weight of the cover soil = γ =	18.0	kN/m^3				
friction angle of the cover soil = φ =	30.0	°	*= 0.52*	*(rad.)*		
cohesion of the cover soil = c =	0.0	kN/m^2			*C= 0*	*kN*
interface friction angle between cover soil and geomembrane = δ =	22.0	°	*= 0.38*	*(rad.)*		
adhesion between cover soil and geomembrane = ca =	0.0	kN/m^2			*Ca= 0*	*kN*

Note: numbers in boxes are input values

numbers in Italic are calculated values

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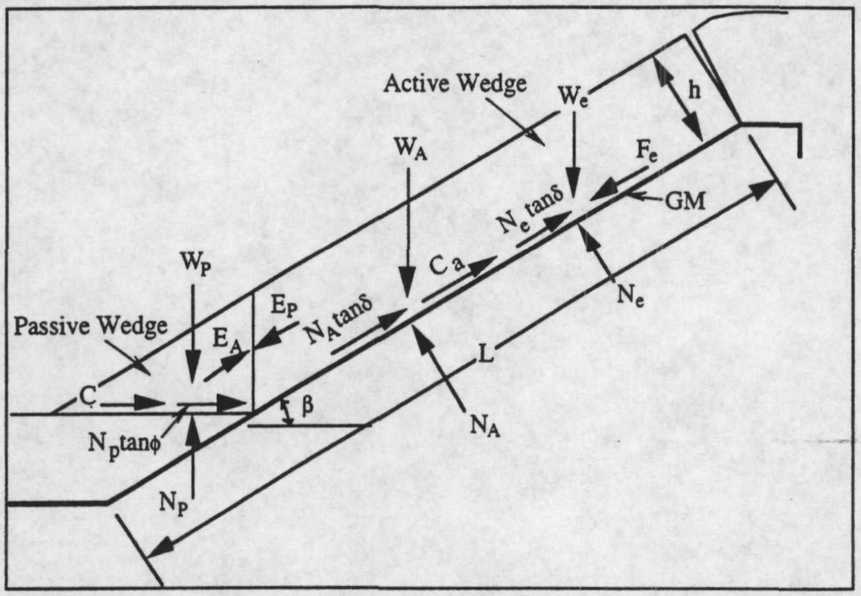
Constructed by: Te-Yang Soong

SOIL SLOPE STABILITY ANALYSIS - WORKSHEET #2(a)

Uniform Cover Soil Thickness with the Incorporation of Equipment Loads



Calculation of FS

Active Wedge:

Wa= 156.6 kN

Na= 148.6 kN

Passive Wedge:

Wp= 2.7 kN

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a= 73.1

b= -104

c= 17.0

FS= 1.24

thickness of cover soil = h =	0.30	m		
cover soil slope angle beneathe the geomembrane = β =	18.4	°	*= 0.32* (rad.)	
finished cover soil slope angle = ω =	18.4	°	*= 0.32* (rad.)	
length of slope measured along the geomembrane = L =	30.0	m		
unit weight of the cover soil = γ =	18.0	kN/m^3		
friction angle of the cover soil = φ =	30.0	°	*= 0.52* (rad.)	
cohesion of the cover soil = c =	0.0	kN/m^2		*C= 0 kN*
interface friction angle between cover soil and geomembrane = δ =	22.0	°	*= 0.38* (rad.)	
adhesion between cover soil and geomembrane = ca =	0.0	kN/m^2		*Ca= 0 kN*

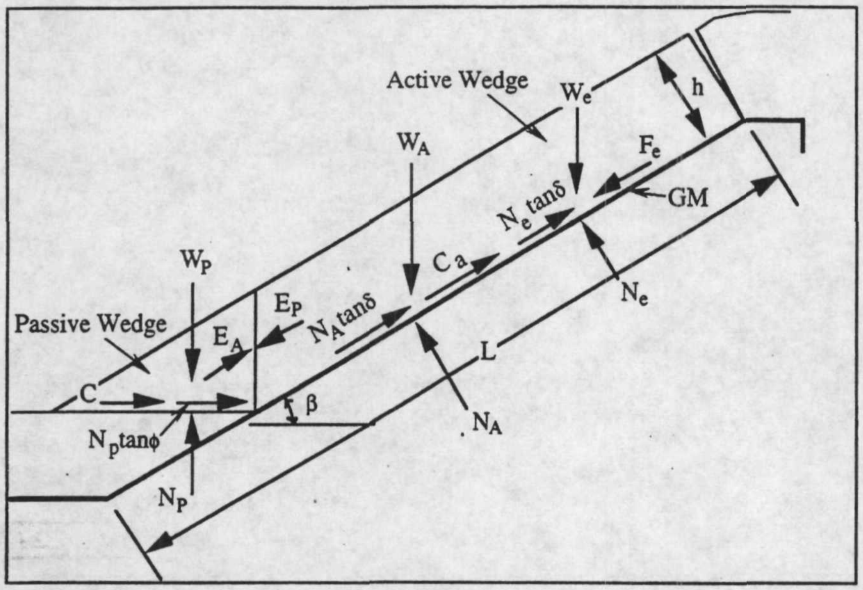
thickness of cover soil = h =	0.30	m	*b/h= 2.0*
equipment ground pressure(= wt. of equipment/(2×w×b)) = q =	30.0	kN/m^2	*We=q × w = 87.3*
length of equipment track = w =	3.0	m	*Ne=Wecos β = 82.8*
width of equipment track = b =	0.6	m	*Fe=We × (a/g) = 0.0*
influence factor at geomembrane interface = I =	0.97		
acceleration of the bulldozer = a =	0.00	g	

Note: numbers in boxes are input values

numbers in Italic are calculated values

SOIL SLOPE STABILITY ANALYSIS - WORKSHEET #2(b)

Uniform Cover Soil Thickness with the Incorporation of Equipment Loads



Calculation of FS

Active Wedge:

Wa= 156.6 kN

Na= 148.6 kN

Passive Wedge:

Wp= 2.7 kN

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a= 88.3

b= -107

c= 17.0

FS= 1.03

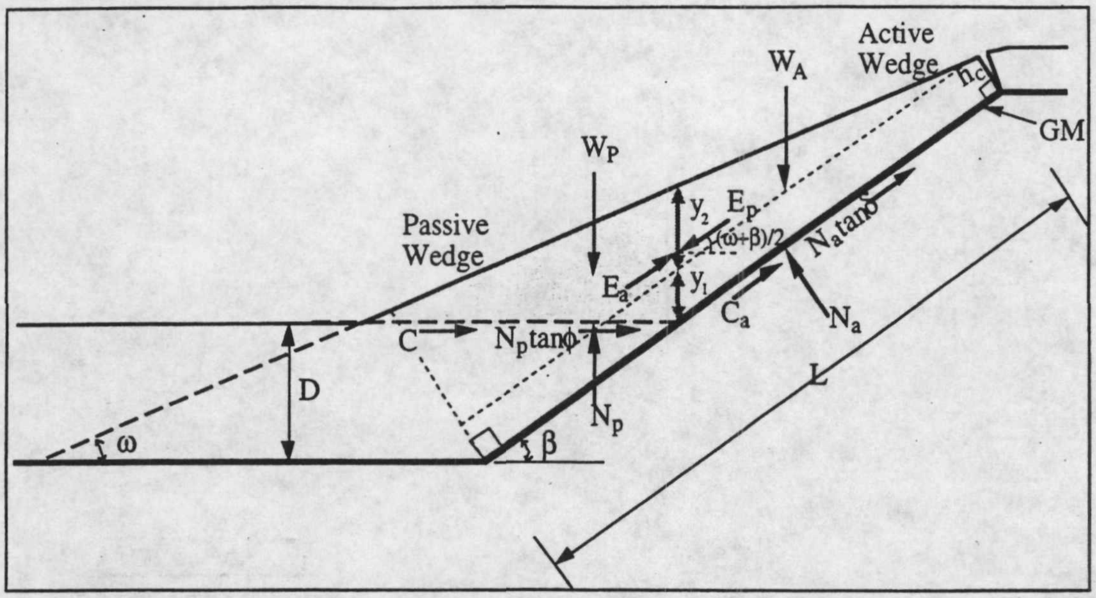
thickness of cover soil = h =	0.30	m		
cover soil slope angle beneathe the geomembrane = β =	18.4	°	*= 0.32*	*(rad.)*
finished cover soil slope angle = ω =	18.4	°	*= 0.32*	*(rad.)*
length of slope measured along the geomembrane = L =	30.0	m		
unit weight of the cover soil = γ =	18.0	kN/m^3		
friction angle of the cover soil = φ =	30.0	°	*= 0.52*	*(rad.)*
cohesion of the cover soil = c =	0.0	kN/m^2		*C= 0 kN*
interface friction angle between cover soil and geomembrane = δ =	22.0	°	*= 0.38*	*(rad.)*
adhesion between cover soil and geomembrane = ca =	0.0	kN/m^2		*Ca= 0 kN*
thickness of cover soil = h =	0.30	m		*b/h= 2.0*
equipment ground pressure(= wt. of equipment/(2×w×b)) = q =	30.0	kN/m^2		*We=q × w × I= 87.8*
length of equipment track = w =	3.0	m		*Ne=Wecos β = 82.8*
width of equipment track = b =	0.6	m		*Fe=We × (a/g) [illegible]= ~~16.1~~ 16.6*
influence factor at geomembrane interface = I =	0.97			
acceleration of the bulldozer = a =	0.19	g		

Note: numbers in boxes are input values

numbers in Italic are calculated values

SOIL SLOPE STABILITY ANALYSIS - WORKSHEET #3

Tapered Cover Soil Thickness



Calculation of FS

Active Wedge:

Wa= 391.2 kN

Na= 371.2 kN

Passive Wedge:

Wp= 63.4 kN

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a= 37.0

b= -64

c= 8.6

FS= ~~1.57~~ 1.02

thickness of cover soil at crest of the slope = hc =	0.15	m			
thickness of cover soil at bottom of the landfill = D =	0.30	m			
cover soil slope angle beneathe the geomembrane = β =	18.4	°	*= 0.32*	*(rad.)*	
finished cover soil slope angle = ω =	16.0	°	*= 0.28*	*(rad.)*	
length of slope measured along the geomembrane = L =	30.0	m			

y2= 1.26 (m)

y1= 0.16 (m)

(ω+β)/2= 0.300 (rad.)

(= 17.2 °)

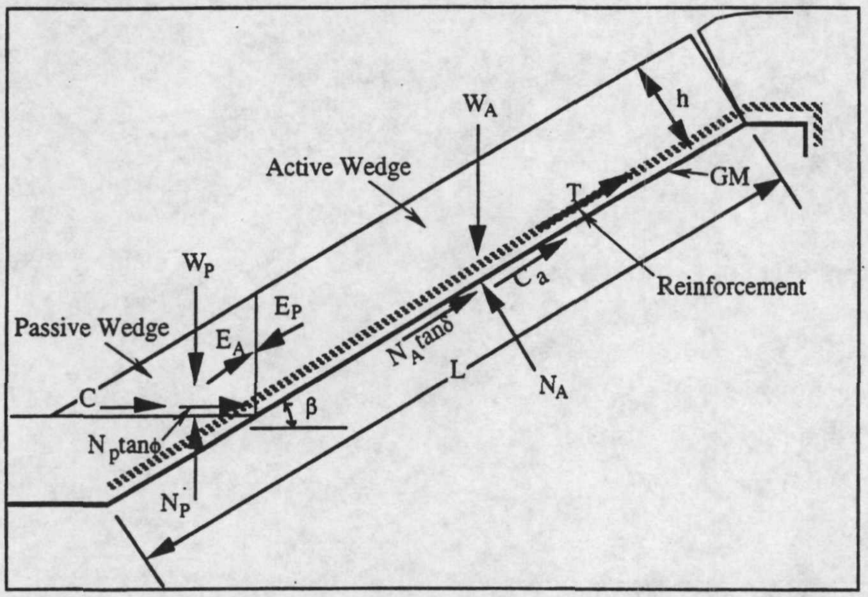
unit weight of the cover soil = γ =	18.0	kN/m^3			
friction angle of the cover soil = φ =	30.0	°	*= 0.52*	*(rad.)*	
cohesion of the cover soil = c =	0.0	kN/m^2			*C= 0 kN*
interface friction angle between cover soil and geomembrane = δ =	22.0	°	*= 0.38*	*(rad.)*	
adhesion between cover soil and geomembrane = ca =	0.0	kN/m^2			*Ca= 0 kN*

Note: numbers in boxes are input values

numbers in Italic are calculated values

SOIL SLOPE STABILITY ANALYSIS - WORKSHEET #4

Uniform Cover Soil Thickness with Veneer Reinforcement



Calculation of FS

Active Wedge:

Wa= 156.6 kN

Na= 148.6 kN

Passive Wedge:

Wp= 2.7 kN

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a= 11.8

b= -21

c= 3.5

FS= 1.57

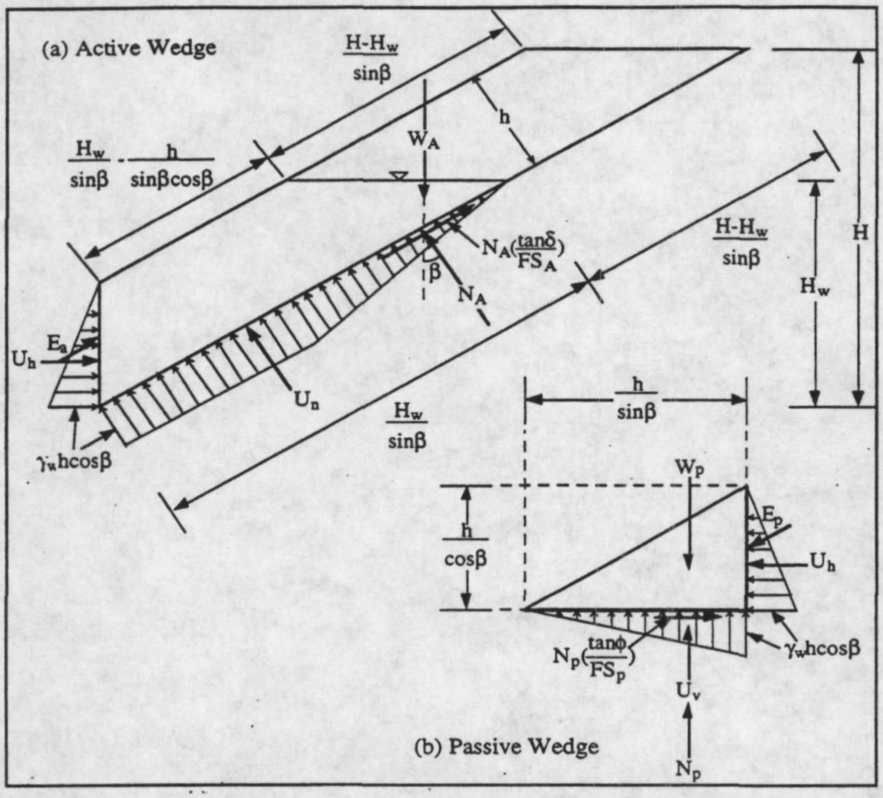
thickness of cover soil = h =	0.30	m				
cover soil slope angle beneathe the geomembrane = β =	18.4	°	*= 0.32*	*(rad.)*		
finished cover soil slope angle = ω =	18.4	°	*= 0.32*	*(rad.)*		
length of slope measured along the geomembrane = L =	30.0	m				
unit weight of the cover soil = γ =	18.0	kN/m^3				
friction angle of the cover soil = φ =	30.0	°	*= 0.52*	*(rad.)*		
cohesion of the cover soil = c =	0.0	kN/m^2			*C= 0*	*kN*
interface friction angle between cover soil and geomembrane = δ =	22.0	°	*= 0.38*	*(rad.)*		
adhesion between cover soil and geomembrane = ca =	0.0	kN/m^2			*Ca= 0*	*kN*
ultimate (manufactured) value of reinforcement strength = Tult =	40.0	kN/m				
partial FS for installation damage = (FS)ID =	1.3					
partial FS for creep = (FS)CR =	2.4					
partial FS for chemical/biological degradation = (FS)CBD =	1.3				*T reqd =*	*10.0 kN/m*

Note: numbers in boxes are input values

numbers in Italic are calculated values

SOIL SLOPE STABILITY ANALYSIS - WORKSHEET #5(a)

Seepage Forces with Horizontal Seepage Buildup



Calculation of FS

Active Wedge:

WA =	*172*	*kN*
Un =	*40.5*	*kN*
Uh =	*0.44*	*kN*
NA =	*123*	*kN*

Passive Wedge:

WP =	*3.16*	*kN*
Uv =	*1.33*	*kN*

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a=	*51.7*
b=	*-58*
c=	*9.0*

FS= 0.93

---

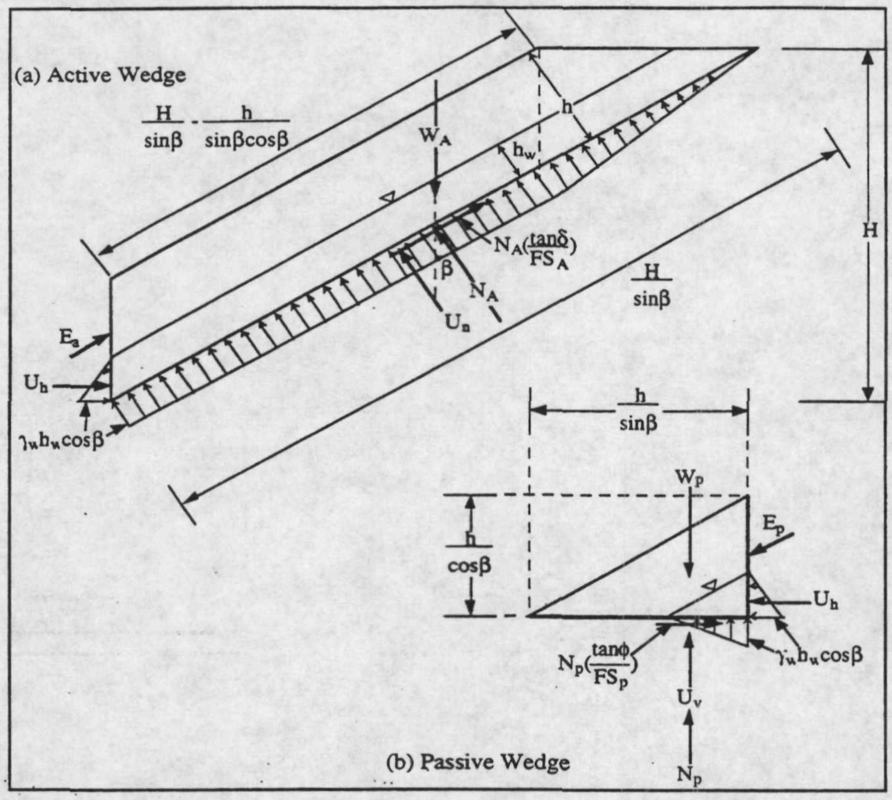
thickness of cover soil = h =	0.30	m	
length of slope measured along the geomembrane = L =	30.0	m	
cover soil slope angle beneathe the geomembrane = β =	18.4	°	*= 0.32 (rad.)*
vertical height of the slope measured from the toe = H =	*9.5*	*m*	
horizontal submergence ratio = HSR =	0.50		
vertical height of the water surface measured from the toe = Hw =	*4.7*	*m*	
dry unit weight of the cover soil = γ *dry* =	18.0	kN/m^3	
saturated unit weight of the cover soil = γ *sat'd* =	21.0	kN/m^3	
unit weight of water = γ *w* =	9.81	kN/m^3	
friction angle of the cover soil = φ =	30.0	°	*= 0.52 (rad.)*
interface friction angle between cover soil and geomembrane = δ =	22.0	°	*= 0.38 (rad.)*

Note: numbers in boxes are input values

numbers in Italic are calculated values

SOIL SLOPE STABILITY ANALYSIS - WORKSHEET #5(b)

Seepage Forces with Parallel-to-Slope Seepage Buildup



Calculation of FS

Active Wedge:

WA =	*173*	*kN*
Un =	*41.5*	*kN*
Uh =	*0.11*	*kN*
NA =	*122*	*kN*

Passive Wedge:

WP =	*2.82*	*kN*
Uv =	*0.33*	*kN*

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a=	*51.7*
b=	*-58*
c=	*9.0*

FS= 0.93

thickness of cover soil = h =	0.30	m		
length of slope measured along the geomembrane = L =	30.0	m		
cover soil slope angle beneathe the geomembrane = β =	18.4	°	*= 0.32*	*(rad.)*
vertical height of the slope measured from the toe = H =	*9.5*	*m*		
parallel submergence ratio = PSR =	0.50			
depth of the water surface measured from the geomembrane = hw =	*0.15*	*m*		
dry unit weight of the cover soil = γ *dry* =	18.0	kN/m^3		
saturated unit weight of the cover soil = γ *sat'd* =	21.0	kN/m^3		
unit weight of water = γ *w* =	9.81	kN/m^3		
friction angle of the cover soil = φ =	30.0	°	*= 0.52*	*(rad.)*
interface friction angle between cover soil and geomembrane = δ =	22.0	°	*= 0.38*	*(rad.)*

Note: numbers in boxes are input values

numbers in Italic are calculated values

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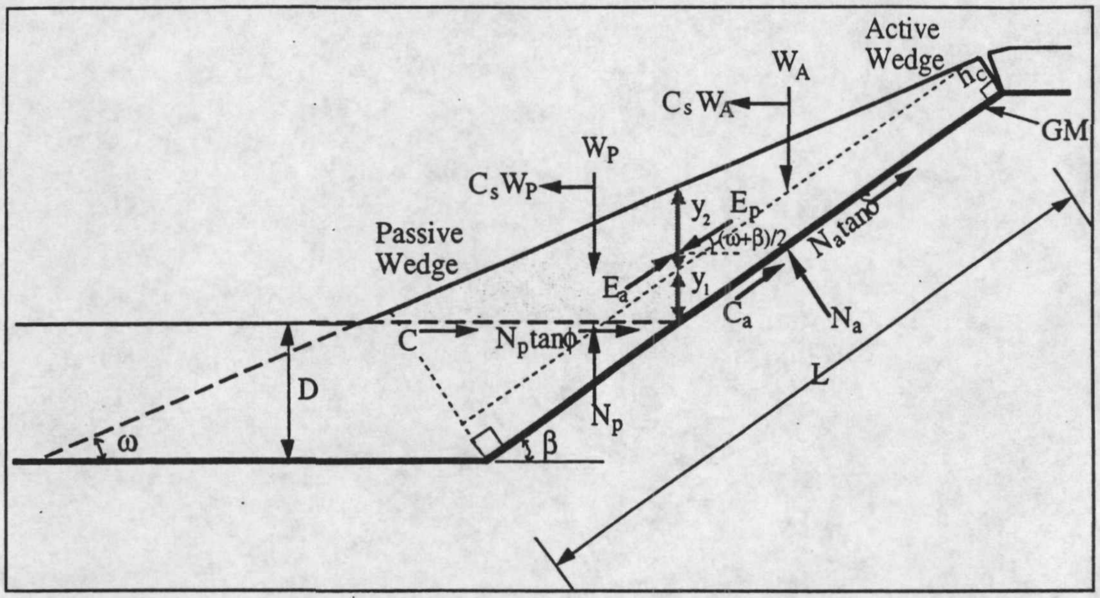
SOIL SLOPE STABILITY ANALYSIS - WORKSHEET #6

Uniform and/or Tapered Cover Soil with Consideration of Seismic Forces



Calculation of FS

Active Wedge:

Wa=	*156.6*	*kN*
Na=	*148.6*	*kN*
Ca=	*0.0*	*kN*

Passive Wedge:

Wp=	*2.7*	*kN*
C=	*0.0*	*kN*

$$FS = \frac{-b + \sqrt{b - 4ac}}{2a}$$

a=	*59.6*
b=	*-67*
c=	*10.4*

FS= 0.94

(Note: input ω equals to β for uniform cover soil thickness)

thickness of cover soil at crest of the slope = hc = 0.30 m

thickness of cover soil at bottom of the landfill = D = 0.30 m

cover soil slope angle beneathe the geomembrane = β = 18.4 ° *= 0.32 (rad.)*

finished cover soil slope angle = ω = 18.4 ° *= 0.32 (rad.)*

length of slope measured along the geomembrane = L = 30.0 m

y2=	*0.00*	*(m)*
y1=	*0.32*	*(m)*
(ω+β)/2=	*0.321*	*(rad.)*
(=	*18.4*	*°)*

unit weight of the cover soil = γ = 18.0 kN/m^3

friction angle of the cover soil = ϕ = 30.0 ° *= 0.52 (rad.)*

cohesion of the cover soil = c = 0.0 kN/m^2

interface friction angle between cover soil and geomembrane = δ = 22.0 ° *= 0.38 (rad.)*

adhesion between cover soil and geomembrane = ca = 0.0 kN/m^2

seismic coefficient = Cs = 0.10 g

Note: numbers in boxes are input values

numbers in Italic are calculated values